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UNITED STATES DEPARTMENT OF AGRICULTURE

BULLETIN No. 304

Contribution from the Office of Public Roads and Rural Engineering
LOGAN WALLER PAGE, Director

Washington, D. C. PROFESSIONAL PAPER November 19, 1915

[Revision of Office of Experiment Stations Bulletin 243]

LAND DRAINAGE BY MEANS OF PUMPS

By

S. M. WOODWARD, Drainage Engineer

Revised with special reference to the Upper Mississippi Valley by C. W. OKEY, Senior Drainage Engineer

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INTRODUCTION.

The drainage of low-lying lands by the use of pumping machinery to lift the drainage water over levees into adjacent streams or other drainage channels is a recent development in this country. Along the banks of many of our larger interior rivers considerable areas of bottom land are subject to overflow from the adjacent streams during the high water occurring usually with great regularity throughout the spring and early summer months. Such lands in their native state do not become dry enough to be subject to ordinary cultural operations until well toward the middle of the summer, and

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NOTE.—This bulletin treats in general of the drainage of land which lies so low that it must be protected from overflow by levees and the drainage water pumped out of the protected area owing to lack of gravity outlet. The bulletin has special reference to low-lying bottom lands along the larger streams of the upper Mississippi Valley.

hence can not generally be used for growing ordinary crops. In this natural condition they are, therefore, good for nothing but pasture, and have accordingly but little agricultural value.

Similarly, extensive marshy areas along our coasts may be for a part of the time above water level, but are subject to overflow at high tide, especially during spring tides and when the river channels which form their natural drainage outlets are in flood. All such low lands are, in general, formed by alluvial deposit and possess a high degree of fertility when relieved of their excess of water and changed from their original saturated condition to a degree of moisture content suitable for ordinary dry-land crops.

As a physical proposition it is possible to reclaim all such areas for agriculture by first constructing levees to keep them from being overflowed, and then, where necessary, by installing pumping plants to pump out from the interior of the district the excess rainfall and such water as may seep through or under the levees in injurious amounts. But as such work is expensive both for the first construction and for its subsequent maintenance and operation, the really difficult and fundamental question for the landowner is whether the value of the land after its reclamation will be sufficient to justify the expenditures necessary to carry out the work.

So long as there exists in a locality any unoccupied or unutilized higher land suitable for agriculture there is little demand for the use of lands lying so low as to require drainage by pumps. But the present high price of agricultural land justifies a heavy expenditure for the conversion to a productive state of areas formerly considered almost valueless, especially in those regions where agricultural land values are particularly high, either on account of unusually favorable natural conditions or the proximity of large centers of population.

EFFECTIVENESS OF DRAINAGE RECLAMATION.

Some areas are so favorably situated that, when once protected by levees from inundation, satisfactory interior drainage without the use of pumps may be secured by a natural or gravity flow of their drainage waters, either through sluice gates or through ditches or natural streams other than those producing the overflow. Examples of such lands are many tidal marshes and the large areas in Arkansas, Mississippi, and northern Louisiana, where the land, protected from the Mississippi River by great levees, has good gravity drainage outlets through other streams.

But much overflowed land is not so fortunately located. Frequently, in addition to a protective levee system, a pumping plant is required for lifting the interior drainage water out over the levee. In fact, numerous cases may be found in which, when the levee system was first constructed, reliance was placed solely upon gravity outlets for disposing of the interior drainage water, but subsequent experience showed that by this means alone satisfactory reclamation of the land could not be secured; hence, a supplementary pumping plant, not contemplated in the original plans, was later installed.

The two characteristics which any agricultural drainage project must possess for its perfect success are, first, effectiveness, and second, profitableness. If an elaborate plan for drainage improvement, when actually carried out, fails to drain the land sufficiently to secure a satisfactory cultivable condition the project is a failure. In fact, it is even worse than a failure, for not only may it mean an actual financial loss to its owners, but as an example may discourage others from undertaking other meritorious projects. So vital is it to secure adequate drainage that if any constructive work at all is undertaken, it would seem that everyone connected with such work would appreciate its importance. Yet almost innumerable instances can be found where drainage improvements have been planned and constructed on a scale entirely too small to secure efficient results, and where those responsible for carrying out the work should have known in advance that the benefits resulting from their misdirected efforts would not be commensurate with the expenditures incurred.

It is characteristic of almost all artificial drainage improvements that, if a certain amount of work is needed, the actual construction of a part only of the whole work will not produce a correspondingly proportionate benefit; that is, if only one-half of the needed work is carried out much less than one-half of the full benefit will be obtained. It is, perhaps, approximately true that the carrying out of three-fourths of the needed work would yield one-fourth of the total benefit. The last one-fourth of the work produces perhaps three-fourths of the benefit. It is on account of this relation that the original planning of all drainage work on a sufficiently comprehensive and adequate scale is so important.

Naturally those who have had no experience in constructing drainage improvements can have little comprehension of the innumerable difficulties and obstacles that may arise in the construction of extensive work and in its operation after completion; and, too often, when they have the responsible charge of such work, they are prone to refuse to accept the advice of others who have had extended experience. There are still intricate problems and baffling uncertainties, numerous enough, in securing drainage in difficult situations, but there would seem to be no excuse nowadays for undertaking important work in violation of fundamental principles of drainage science well known to every experienced worker in this field of engineering.

It is not alone sufficient, however, that the drainage work as constructed shall secure the desired benefit. The work must be done economically and in such a way as to secure the greatest profit to the landowner. Several quite different plans for the drainage of a given district might secure equally well the desired improvement. In such a case that plan should be chosen which will prove cheapest in the long run; and to determine the best and cheapest plan is the special province of the engineer. One of the first steps taken in beginning a large drainage project should be to secure the services of a competent and experienced engineer. Such a man is needed to make the necessary surveys and plans for the work; to make inspections and estimates during construction; and, when a pumping plant is a part of the work, to test the completed plant to see that it complies with the specifications and meets the guaranties of capacity and efficiency. He is needed also to advise as to the most economical and efficient operation of the plant after its completion and as to its management, maintenance, and care.

Drainage by means of pumps has been carried on in European countries for the last 100 years and has been rapidly increasing in this country during the last 25 years. Through this extensive experience, including numerous failures, a considerable amount of knowledge is now available on the subject of the proper arrangement and the requisite capacity of pumping plants. It is the object of this bulletin to discuss the various questions arising in connection with the reclamation of agricultural land by means of pumps in order that the conclusions drawn from experience may be available to all landowners, district officers, and drainage engineers who are interested in this kind of drainage.

DRAINAGE PUMPING IN NORTHERN EUROPE.

Draining with the aid of pumps, or what may be termed "lift drainage," is required for large areas of productive agricultural land in Europe. Notable examples of successful work of this character are found in Holland, eastern England, and in Ireland, while large marsh areas in both northern and southern Italy depend upon pumps for adequate drainage. The development of mechanical means of removing water from the land has, in most instances, been forced upon landowners when gravity drainage failed to give sufficient relief, and for that reason the movement was slow, particularly in the early history of such drainage. The pumps which were first used were the well-known scoop wheel and the Archimedean screws, which were driven by windmills.

The Zuidplas polder of Holland, containing 10,363 acres, lies 22 feet below the level of the River Yssel, into which it is drained. In 1825 the basin was drained by dividing it by a canal into two parts and raising the water into the river by two lifts. The first or lower lift was accomplished by 15 windmills driving 8 Archimedean screws and 7 scoop wheels, while for the upper lift there were 15 windmills driving 10 screws and 5 wheels. These were later supplanted by two pairs of pumping stations, each having scoop wheels and centrifugal pumps, all driven by steam, by which means the drainage of the entire tract is now accomplished.

The great Haarlem Lake of Holland is often referred to as the most remarkable pumping drainage project known. The total area of the tract, which is now surrounded by a canal 37 miles long, is 41,648 acres, 3,011 of which are occupied by roads and main waterways. The work of removing the water in this lake, which was originally 15 feet deep, was completed in 1852, after 39 months of pumping. It is kept drained by three pumping plants of 350 horsepower each, located at opposite sides and at one end of the great basin. The combined capacity of the three plants is nearly 2,000 tons of water per minute raised to a height of approximately 15 feet. The area which was formerly a lake is now traversed by well-improved highways and is occupied by about 20,000 people.

In September, 1913, the Dutch Government authorized the undertaking of a much greater project. It is planned to dike off about 1,000,000 acres of the southerly portion of the Zuider Zee. A little over one-half million acres will be completely reclaimed and made suitable for the production of crops, the remainder will form a reservoir for the discharge of the River Yssel while the tide gates into the untouched portion of the Zee are closed. The work, as planned, will consume 33 years in the construction and will require an appropriation of $130,000,000. The annual revenue from the cultivation of these lands is estimated at $28,000,000 and the population it will support at 300,000. Fifteen steam pumping plants with a combined horsepower of 17,000 will lift the water out of the various divisions of the main district.

The drainage of large portions of the fenlands in eastern England was originally accomplished by means of scoop wheels operated by windmills. It was not an uncommon occurrence for the districts to be inundated because of the failure of the pumps to operate when required to remove the water. Losses from this cause became so great that the several plants were gradually equipped with steam power, which rendered the drainage of the land much more certain than under the old system.

W. H. Wheeler, in his History of the Fens of South Lincolnshire, states that the entire fens known by that name cover about 363,000 acres, that about 85,000 acres are from 6 to 12 feet below high-water level, and that 124,600 acres of this area are drained by steam power. The pump originally used was the large scoop wheel still operated in many of the plants, but the centrifugal pump is gradually being introduced as being better suited to the conditions now prevailing. The same author, in his book entitled "The Drainage of Fens and

Low Lands," enumerates 13 distinct districts having areas varying from 800 acres to 35,000 acres which are now drained by steam plants operating either the improved scoop wheel or the more modern centrifugal pump.

From available data it appears that not less than 203,000 acres of lowland in eastern England depend upon pumps for their drainage. The efficiency of these plants has from time to time been greatly increased until it is now estimated that the average cost of pumping in eastern England is 2 cents per acre for each foot the water is raised.

The conditions which are found in Europe are in many respects different from those we are required to meet in this country. The history of drainage by pumping, however, during the last 100 years shows a gratifying and substantial development of mechanical devices and the utilization of power for draining, and establishes beyond question the practicability of this kind of reclamation. The experience and practice of northern Europe are exceedingly valuable to those who contemplate the design and construction of works of this class, a study of which should prevent us from falling into certain errors that have at times been expensive and discouraging to European enterprise in land development.

PAST EXPERIENCE IN THE UNITED STATES.

In this country, among the plantations in the lowlands along the Gulf of Mexico, pumping has been in vogue for a long time, especially for the drainage of lands devoted to the cultivation of sugar cane. Recently the reclamation of the wet lands in southern Louisiana has progressed rapidly and there are now a large number of projects completed and others in the various stages of construction. The total area thus taken in is about 240,000 acres.

Pumping was first introduced in the bottom lands lying along the Illinois River and the adjacent portions of the Mississippi about 25 years ago, but little was accomplished previous to 1900. Since 1905, however, numerous large plants have been constructed. In the spring of 1915, 19 separate plants along the Illinois were in operation, pumping the drainage water from a total area of about 161;000 acres. These plants have a combined capacity of about 6,300 horsepower and represent an investment of about $500,000. There are under construction three additional plants to provide for the drainage water from about 21,000 acres. With the completion of this work nearly all the available bottom land along the Illinois River will have been reclaimed. For all the improvements constructed in drainage districts as such the expenditure was about $4,800,000.

Along the Mississippi River in the States of Iowa, Missouri, and Illinois the movement is well started, and it is now evident, from the numerous inquiries for information received from these and many other States, that there exists at the present time a very active and widespread interest in this whole question. Between Muscatine, Iowa, and St. Louis there are in operation 10 separate plants, which take the drainage water from about 215,000 acres of land. Other plants are under construction or contract on five districts which have a total drainage area of 128,000 acres. The combined horsepower of the plants on the Mississippi is about 7,000. The cost of the district drainage improvements, including the pumping plants, totals a little over $4,000,000.

The experience of those who undertake projects for pumping drainage water without previous experience in similar work or full and accurate knowledge of what has been accomplished elsewhere shows that invariably they greatly underestimate the magnitude and difficulties of the work contemplated. In this regard their experience is identical with that of those who have been pioneers in other forms of agricultural drainage work. As a consequence the first pumping plants were entirely inadequate in capacity, inefficient in operation, and largely a waste of the money invested in them. In numerous cases they have been added to or entirely replaced by the building of new plants. The officials having in charge the construction of the improvements on the newer districts have profited by the experience of the older districts, and have realized the folly of spending money for inadequate improvements. As a result nearly all of the more recent plants, with their ample and economical machinery, are models of convenient arrangement, permanence, and durability. The decreased cost of operation and the splendid drainage obtained have paid for the increased expenditure many times over.

In the upper Mississippi Valley the various conditions prevailing, such as intensity and distribution of rainfall, length of growing season, and kinds of crops raised, differ widely from those obtaining near the Gulf of Mexico in Louisiana, leading to correspondingly wide variations in recent drainage practice. The former situation will here be taken up in detail, and the planning of districts, the requisite sizes and kinds of machinery, the first cost, and the cost of maintenance will be discussed. The situation in Louisiana is dealt with in a previous publication.[1]

DRAINAGE BY PUMPING IN THE UPPER MISSISSIPPI VALLEY.

In Illinois and the adjacent States the bottom lands which have been reclaimed by pumping are in the heart of the corn belt and possess a heavy, rich, black soil, mixed in places with a varying amount of sand. They are adjacent to thickly populated lands, and in some cases lie within a few miles of large towns. This proximity to some of the most fertile and highest priced agricultural lands in the United States has led to an earlier activity in drainage reclamation than has existed in most other portions of our country. The results of adequate protection and drainage are shown by figure 1 of Plate IV, a view of part of a tract of reclaimed land on the Illinois River.

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[1] U. S. Dept. Agr., Bul. 71, The Wet Lands of Southern Louisiana and Their Drainage.

SIZE OF PUMPING DISTRICTS.

In order that the building of a levee around a tract of overflowed land and the installation and operation of a pumping plant may be carried out economically at a moderate cost per acre of land reclaimed, it is necessary to have a considerable area included in a single project. Hence, ordinarily the land does not belong to a single owner, but rather to a number of individuals. In order that all the interested landowners may work harmoniously together it is necessary, first, that an incorporated drainage district be formed according to the method provided by the specific laws of the particular State relating to this subject. The officers of the district so formed may then proceed to carry out the successive steps necessary in the inauguration of the construction work.

Pumping districts of as small an area as 3,000 acres may profitably be formed in favorable locations; but, in general, the expense per acre is less as the district is larger up to a certain limit beyond which there is no further advantage from increased size. According to experience, about 10,000 acres of reclaimed land seems to be a very satisfactory size for such a district.

However, the boundaries of a district are determined largely by the natural surface conditions. A district will naturally include all the land on one side of the river between the channel and the bluff and extending up and down the valley from one tributary to the next of any size entering the river on the same side. The tendency in recent years has been to make the districts larger and to extend them beyond the so-called natural boundaries—the tributaries to the river—by diverting these streams around the district or by carrying them by means of levees through the district to the river. The internal drainage channels are carried under these diversion channels by means of concrete inverted siphons or culverts. By this increase in area of the lands organized in one district the expenses per acre of organization and administration during construction and operation are materially reduced. The charge per acre for operation of pumping plant is, in general, much less on the larger district. As the valley ordinarily has the same slope as the high-water line of the adjoining river, the lift of the pumps in forcing the drainage water into the river is very rarely increased by extending the district a greater distance along it. On the other hand, smaller districts are being now reclaimed than formerly was the practice. This is due to the increased demand for land, which makes reclamation profitable for small, isolated tracts of valley land whose reclamation had been deferred because of the heavy expense attached. The future will see the reclamation of both larger and smaller districts.

LEVEES.

The protective levee begins at the bluff at one end of the district, follows along the bank of the entering tributary to its junction with the river, thence along the river bank to the other tributary, and up that again to the bluff. Thus, ordinarily, the levee extends continuously around three sides of the district. The correct location of the levee to secure a good foundation and to leave sufficient floodway for the river, and its proper construction of sufficient strength to resist erosion, undermining, and overtopping, are engineering questions of vital importance to the welfare of the district. It was the unfortunate experience of nearly every one of the early levee districts on the Illinois River that, as first constructed, the levees were entirely inadequate in size and strength, with the result that they were broken by floods, their interiors were inundated, and years elapsed before the districts recovered from the injury. With the interior of a district in cultivation the damage from one flooding due to a break in the levee might easily exceed the entire cost of building a levee of ample size to protect the district. This has occurred on a number of the levee districts on both the Illinois and the Mississippi Rivers. The levees on the Warsaw-Quincy District were originally built and later repaired and strengthened at a total cost of about $500,000. This district was flooded in four different years, and each time the estimated damage was over $600,000. By spending the proper amount of money originally a saving of at least $1,500,000 could have been made. The total cost of the levees, including first cost, repairs, and enlargements on the Sny Island District was about $1,125,000. The estimated damage of five years of flooding between 1876 and 1903 was approximately $2,700,000. Similar damage has resulted in many levee districts. The older a district becomes, the greater the damage which may result from a break in the levee. In the construction of the levees around new districts the need of an ample margin of safety is now generally admitted, and many of the older districts are enlarging levees which have withstood record high-water stages in the adjoining river. Officials and landowners in levee districts should make certain that the levee is properly designed, located, constructed, and maintained.

LOCATION.

The location of the levee will influence its design, construction, and maintenance. Intelligent location will aid in avoiding poor levee material and bad foundation. Every effort should be made to avoid locating the levee where it will be exposed to strong currents or direct wave action from the main river channel. Many levee failures have occurred because of poor material, unstable foundation, or erosion. It is usually poor economy and often disastrous to place the levee too near the river channel in order to reclaim a few more acres of land. The ground is quite often lower nearer the river channel, and besides the danger of failure due to erosion, the maintenance expense on levees exposed to wave and current action is always heavy. Government engineers in charge of the Mississippi River work now require that the borrow pits made in levee construction be at least 100 feet from the bank of the river, thus making the minimum distance from the levee to the bank not less than about 150 feet. Consideration must be given to the question of leaving outside the levee a sufficiently wide flood channel. The conclusion reached in any particular case will depend upon the conditions existing upon the opposite banks of the stream, and no definite rules can be stated.

DESIGN.

The height of the levees on districts along the Illinois and Mississippi Rivers varies all the way from zero at their ends against the high ground to 20 feet in places, but they are in general between 8 and 12 feet high. These heights have been fixed with reference to the height of the flood stages in the rivers above the land upon which the levee is located. The top of the levee should be at least 3 feet above highest expected stage.

The dimensions which a levee must have in order that it may be durable and sufficiently strong depend upon the kind of material used, the method of construction, and the nature of the exposure of the levee to the action of waves and currents. A tenacious, clayey soil or gumbo will in general make the best levee. Experience has shown that the least dimensions which a finished and compacted levee should have are a width on top of 6 feet and a slope on each side of 3 horizontal to 1 vertical. Where the location is such that the water remains against the outer side of the levee for only short periods of time, and where the material is exceptionally good, a total slope of 5 to 1 might be divided equally between the two sides. If the material used is very sandy, or if the bank is to be exposed to wave action or strong current, flatter slopes must be used. In this connection it has been the experience on many districts that while coarse, pure sand will erode very readily it will not give trouble from settlement or sliding on the land side during high water. On the other hand, a mixture of fine sand and clay often will become saturated and semiliquid during prolonged high water and disastrous slides may occur very suddenly. By making the slopes of the levees not steeper than 3 to 1 the all-important maintenance of the levees is much facilitated.

Where good foundation can not be secured for the levee it often becomes necessary to build a sublevee along the land-side toe of the levee slope. This sublevee would usually start at a point on the land-side slope about 8 feet below the grade line of the main levee, run back with a slope of 20 to 1 for about 20 feet, and then down to the surface of the land with a 3 to 1 slope. This type of construction will aid very materially if there is danger of the levee material sliding, it will make available a large quantity of earth for emergency work on the top of the levee, and will also provide a very suitable location for a road.

CONSTRUCTION.

Levees are built with floating dredges equipped with dipper or grab bucket; with land machines usually equipped with drag-line buckets, but sometimes with grab buckets; and with scrapers. Local conditions will usually determine the type of machine to use, but the majority of levees are now constructed with the land machine equipped with a drag-line bucket. If the material is put up wet, there will be very little subsequent settlement, but if the material is dry or if scrapers are used, the original height of the levee must exceed by at least 10 per cent the height required for the permanent levee. In any case the material used to form the levee should all be taken from the river side of the embankment and a clean berm at least 20 feet wide left between the toe of the slope and the nearest point of the borrow pit. The latter must be kept shallow on the side nearest the levee, with a side slope not steeper than the levee slope, so as not to undermine the embankment. In no case should the excavation cross the imaginary line formed by extending the line of the outer slope of the levee down below the surface of the ground. Where a current along the outside slope of the levee is at all likely, transverse strips of earth of a width of 15 or 20 feet should be allowed to remain across the borrow pit at a spacing of about 500 feet. This will reduce the scouring action of river currents and aid the deposition of sediment in the pits. Land machines adapt themselves to this method of excavation, while floating equipment does not. Recently hydraulic dredges have been used very successfully in building levees. This method is especially desirable where the foundation and material at hand are both poor, a combination of conditions which often occurs. The material usually placed into levees by hydraulic dredges is coarse river sand, taken at some convenient point in the river channel. By this method the base of the levee and the surface between the levee and the river bank are left undisturbed and the difficulty of getting a land machine to work across bad foundation is avoided. Experience has shown such levees to be satisfactory. Hydraulic-fill sand levees are also very desirable in crossing the mouths of sloughs which enter the river channel. Plate I shows hydraulic-fill levees during construction and after completion. The completed levee shown has a top width of about 8 feet, side slopes of 3 to 1, and a height of 12 to 15 feet. Plate II shows a levee constructed by a drag-line excavator.

As in the construction of any other structure, the foundation of a levee is the prime consideration. Unless the material in the foundation is suitable, and unless proper precautions are taken, the levee may fail in spite of the best of construction work on the levee itself. Insufficient care in preparing the foundation for levees has been the most frequent cause of subsequent disaster. Before constructing the levee, the entire area to be occupied by it should be cleared of all vegetation, and all stumps and roots to a depth of 4 feet should be grubbed up. The entire surface should then be plowed, with a large dead furrow in the center. This treatment may suffice if the material in the base is firm and dense, if the levee is only moderately high, and if the water will not stand against the levee for more than a few days at a time. If, however, the material in the base is not of the best, or if the levee is more than 10 feet high, or if the water in the river will remain against the levee for periods longer than 10 days at a time, a muck ditch should always be dug along the center line of the levee. This ditch should be at least 3 feet deep in any case, and always deep enough to cut off any possible seepage. It should be at least 3 feet wide on the bottom and be filled with the best material obtainable, preferably a clayey mixture well compacted. No vegetable matter should be permitted in any of the material used to form the levee. If the subsoil in the borrow pits is clay, this material should be placed in the center of the levee to form a continuous core wall from the muck ditch upward to the top of the levee. The top soil, being thus placed on the slopes of the levee, will facilitate the growth of grass.

MAINTENANCE.

The care of levees subsequent to their completion is an important matter too often neglected. The slopes should be smoothed and as soon as practicable a good growth of grass should be secured. For this purpose bluegrass, Bermuda grass, redtop, timothy, and clover are all used. A good growth of sod will hold the soil in place and prevent erosion. A mowing machine can be used on a 3 to 1 slope, but scarcely on a steeper slope. On a steeper slope, too, beating rains are likely slowly to wash out the soil and thus gradually reduce the height of the cross section. All weeds and brush on a levee should be cut at least twice during the growing season, as they are particularly harmful, loosening and disintegrating the soil by their roots. Sometimes the top of a levee is used as a road, but this practice is not to be commended, for the wagon wheels cut off the corner of the top and the ruts likely to be formed grow rapidly to such dimensions as seriously to affect the height and efficiency of the levee. The proper place for a road along a levee is on the level ground just inside its inner toe. Burrowing animals of numerous sorts are a constant menace to the integrity of a levee, and they should be assiduously hunted and driven away. Frequently levees are pastured. This has the advantage of keeping down the vegetation, keeping the soil compacted, and driving away burrowing animals. The slight damage to a levee which may result from using it as a pasture is easily observed and repaired and is probably counterbalanced by the good results of the practice. Where the levee is exposed to strong wave action and current erosion, special means must be taken to prevent serious damage during high-water stages. The growth of brush and trees on the edge of the berm next the borrow pits and on the strip of land between the borrow pit and the river should be encouraged. If there is no such natural protection, willows should be planted. In the period before the willows reach sufficient size to be effective some form of artificial protection should be used. Tight board fences are often used for protection against wave wash; a layer of rock, or rock with willow mattress, is often used where the river is cutting the bank and threatening the foundation of the levee. In any case the danger is too serious to be met with halfway measures.

Constant attention and expense, the latter not large, are required for the proper maintenance of a levee. In the long run it will be found infinitely more profitable to incur this small regular expense than to invite disaster during some period of unusually high water by permitting the levee to deteriorate to the danger point. Proper maintenance will be found the best economy in the end.

For a more complete discussion of levee construction, methods of protecting inadequate work in emergencies, and the repair of levees after they have been overtopped and broken, the reader should consult a previous publication.[1]

INTERIOR DRAINAGE DITCHES.

An excessive and injurious amount of drainage water may accumulate within the district from three sources: (1) Rainfall, (2) run-off from higher lands draining naturally into the district, and (3) seepage under the levees when the river level is higher than the surface of the land inside the levee. This water must be collected in ditches and led to some suitable point near the levee where the pumps may lift it over the levee and discharge it into the river. The amount of rainfall to be removed by pumping depends upon the amount and distribution of the seasonal precipitation.

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[1] U. S. Dept. Agr., Office Expt. Stas. Bul. 158, Separate 9, Report of Drainage Investigations for 1904.

The quantity of run-off from the bluff lands will depend upon their topography and the natural watershed boundaries; the task of disposing of the run-off is often one of the most serious problems to be solved in the planning of a district. The amount of water thus entering the district should be reduced as much as possible by proper location of the levee and by a diversion ditch wherever the latter is feasible. The rate of run-off from the higher lands is likely to be several times as great as that from the bottom lands, so that the diversion of the run-off from a relatively small amount of hill land may greatly reduce the necessary capacities of the internal drainage ditches and pumping plant and diminish the annual operating charges.

Owing to their high velocities the hill streams carry a great amount of silt, and as the slopes of the channels in the bottom lands are very flat the velocity of flow is reduced and the larger percentage of the silt is deposited soon after the water leaves the hills. On many of the districts along the Illinois and the Mississippi Rivers the silting of the interior drainage ditches by waters from hill streams has entailed a heavy expense for its removal. During 1914 a contract was let by the Hillview Levee and Drainage District to remove an estimated total of 90,000 cubic yards of silt from the interior drainage ditches at a unit price of 18 cents per cubic yard. On the Mississippi River the surveys showed that on the newly organized Indian Grave Drainage and Levee District certain natural channels had been silted by hill streams to such an extent that it was considered necessary to construct new artificial channels, although at a very recent date the natural streams were of ample cross section to serve as main drainage channels for the district. It is estimated that because of this silting it will be necessary to excavate several additional miles of channel at a cost of from $30,000 to $40,000. It is evident that the damage which the silting of interior channels may cause will justify in most cases a considerable expenditure for protection.

The only district known on either of the above rivers where diversion has been tried on an extensive scale for any considerable length of time is the Coal Creek Levee and Drainage District near Beardstown, Ill. (See fig. 1.) A large diversion ditch and levee were constructed about 10 years ago along the base of the bluff for a distance of about 4 miles, from the point where Coal Creek debouches from the hills to the lower end of the district. The levee is on the lower side of the ditch; that is, on the side toward the district. Not only is the flow of Coal Creek received by the diversion ditch, but several smaller streams also flow into it. This ditch, though sufficiently large at first, became too small by being filled with sediment brought down from the hill during floods, so that the various streams overtopped the levee and flowed into the district. In 1909 another ditch and levee were completed, but these differ from the previous construction by being so located as to include a settling reservoir opposite the mouth of each stream. These reservoirs receive the sand and sediment brought down from the hills, and thus prevent the ditch from becoming clogged. The reservoirs were planned to be of such size that they would not become filled with sediment for many years. Two of them are, respectively, 5 and 20 acres in extent, while two are of 6 acres each and three are of 4 acres each. Figure 2 of Plate IV is a view looking west-northwest across Hood Basin, the largest of the reservoirs. Across the basin may be seen a part of the embankment of the original diversion ditch which failed. The

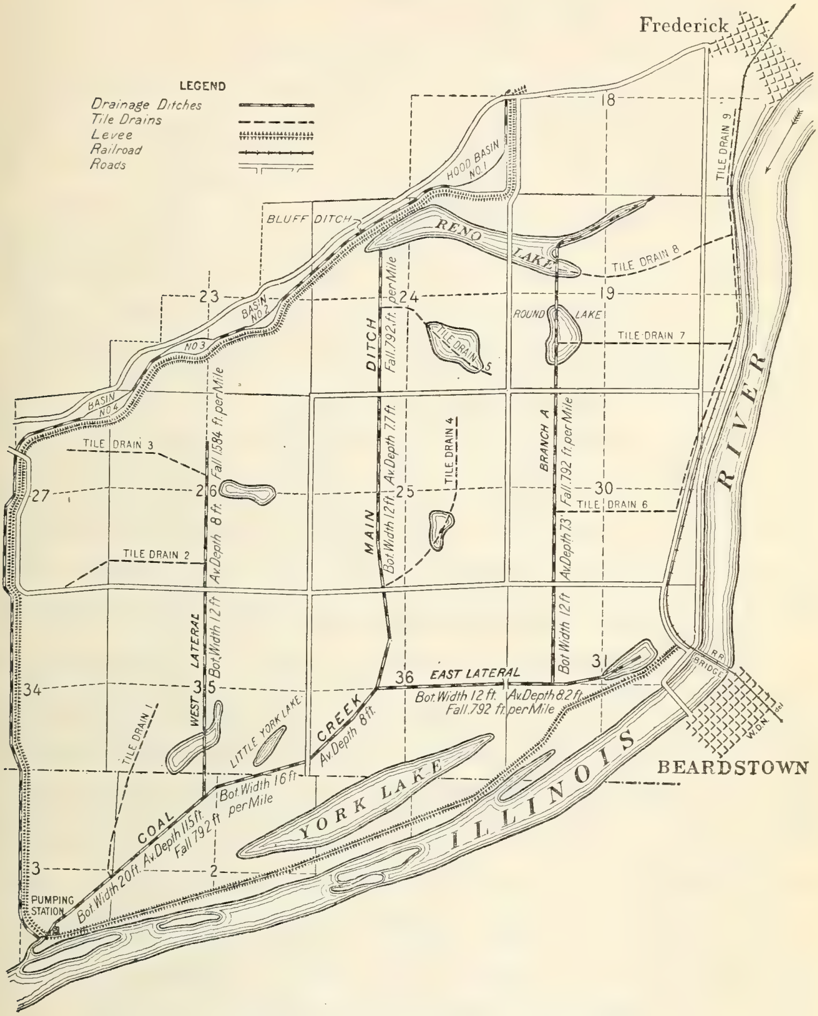

FIG. 1.—Map of Coal Creek Levee and Drainage District, Illinois.

hills in the background show the nature of the country drained by Coal Creek. The last-built ditch and its reservoirs have now been in use for six years and an inspection made in the latter part of 1914 indicated that they would still be effective for a number of years. No other district has installed so complete a system of silting basins in connection with diversion ditches. The Eldred and the Crane Creek Districts on the Illinois River and the Ellsberry and the Des Moines County No. 1 Districts on the Mississippi River have constructed diversions for the hill drainage, though not all of them were able to divert the hill drainage in its entirety. The locations of these districts and the details of their improvements will be found in the summary of all districts given at the end of this bulletin.

In constructing diversion ditches the experience of the Coal Creek District should be kept in mind and ample provision made for the deposition of silt before it reaches the diversion channel proper, otherwise the usefulness of the latter will be of very short duration. Even when silting basins are provided it should be remembered that they will eventually become filled, and they should therefore be so located that other basins can later be constructed to receive the silt from the same stream without incurring needless expense due to the improper location of the first basin. Local conditions will determine the plan to be followed, but the subject merits the most careful study. While in this country work of this nature is just starting, in England and Holland diversion ditches to prevent drainage water from flowing from higher lands onto lower areas, to the great damage of the latter, have been in systematic use for a long time.

If the levees are properly constructed, as heretofore explained, the amount of seepage water passing under them is ordinarily so small as to be negligible, but if the soil or the subsoil of the district be very sandy, or if the levees have been carelessly constructed upon an improper foundation, the seepage may accumulate in sufficient quantity to be troublesome. Ordinarily, if land near the levee is injured by seepage, the difficulty may be corrected by laying a line of tile parallel with the levee and 50 to 100 feet from the toe of the slope.

The determination of the proper size and arrangement of the interior drainage ditches for a pumping district is not fundamentally different from the planning of a gravity ditch system for an ordinary district. The same hydraulic principles apply to the one as to the other, but in some details essential modifications are necessary. Ordinarily river-bottom land has a somewhat uneven surface, varying in elevation by several feet, and lies in long ridges and depressions, which latter bear local names as lakes, sloughs, or creeks. The ridges may be quite sandy, while the lakes contain a heavy black soil. To fit all the land for cultivation the drainage system should be planned to keep the ordinary ground-water level at least 3 feet below the surface of the ground in the lowest spots that are to be cultivated. This is necessary, not only for the purpose of providing room for the root systems of growing crops, but also as a margin of safety by furnishing a reservoir capacity in the drained soil for the temporary storage of water after an unusually heavy period of rain.

In some localities there are sloughs so low that to drain them would require undue expense. In such a case it is better to reserve these areas for use as reservoirs in connection with the ditch system, with the expectation that they will fill with water during heavy rains and store it until it is gradually removed by the operation of the pumps. This tends to reduce the maximum capacity required of the pumping plant of the district concerned. Some districts on the Mississippi River have as much as 7 per cent of their total area occupied by such storage basins, but other districts have none at all, and such storage areas do not seem to be essential in regions where the natural topography of the ground surface is not favorable for their use.

The main ditch and its branches should traverse the lowest areas in the district, following the land subdivisional lines where practicable. This will secure surface drainage in the lowest areas, where it is most needed, and also prove cheapest because the amount of excavation will be a minimum. Cutting ditches through the ridges should be avoided as much as possible, especially where these are sandy, on account of the great difficulty of making and maintaining a ditch in sandy soil owing to the trouble caused by the caving banks. At times the previously existing natural drainage channels, when suitably cleaned, may be made to form a considerable part of the ditch system.

Sometimes the district as a whole has very little surface slope, and it becomes necessary to dig the main canal deeper as it nears the pumping plant in order to keep the water surface parallel with the ditch bottom. Having determined the point above which the water should not rise in the portion of the district remote from the pumping plant, and knowing the capacity of the plant, the slope necessary in a ditch of given size to bring the water to the pumping plant fast enough to keep it operating continuously can be calculated. A small ditch will deliver the water at a lower point at the pumping plant than will a larger one, as the slope will have to be greater for the same discharge. This will mean increased power for driving the pumps. The size of ditch and of pumping plant should be mutually adjusted to secure a minimum operating cost, including interest on investment, depreciation, and expense of operation for the pumping plant, and interest and depreciation on the ditch. The foregoing is based on the fact that the capacity of pumping plant used in this section is nearly uniform. At times of heavy rainfall the ditch will be filled with water and for a short time will deliver more water than is necessary to keep the plant operating; at other times the low-water flow of the ditch will be sufficient to warrant the plant operating only a small portion of the time. This feature will be discussed in more detail under the heading, "Necessary capacity of pumping machinery."

The greatest amount of sediment is likely to be washed into the main ditches during a heavy storm when the ditches are bank full. It should be noticed that at such a time there is an important difference between the condition of these ditches and that existing in main ditches of gravity drainage systems. In the latter, when more than normally full, the velocity is a maximum, and hence there is the least possible tendency toward the deposition of sediment in the ditches. But in a pumping district, on the contrary, when the main ditches are unusually full, the velocity will be low, and hence sediment may be easily deposited. As a rule, the velocity in these ditches will always be so low that they are not self-cleaning. On this account, as well as on account of the drawing down of the level in the ditches by the pumps, such ditches should be of unusually generous dimensions. The engineer will wisely give particular attention to the designing of the ditches to insure ample depth and breadth to provide the requisite capacity. In general, it is probable that the main ditches should be dug from 8 to 10 feet deep. Figure 1 of Plate V is a view of the main ditch, Coal Creek Levee and Drainage District, Illinois, which was constructed in 1909 with a dipper dredge having side spuds. The view was taken before the ditch was completed at the lower end. The water level as shown is about 4 feet below the ground surface and about 4 feet above the normal level after the ditch was completed. Figure 2 of Plate V is a view looking up Branch A of east lateral, from near the outlet of tile drain No. 7. An inspection of these ditches at the end of 1914 showed them to be in very good condition and remarkably free from deposits of silt. This very desirable condition of the ditches is largely due to the complete diversion of silt-bearing waters from the hills. Other districts already in operation would do well to investigate the advisability of treating their hill drainage in a similar manner.

Experience shows that to put bottom land into perfect agricultural condition considerable tile drainage is necessary. This has been demonstrated on all the districts along the Illinois and Mississippi Rivers, and landowners who have already been put to considerable expense for the main drainage of the district can not afford to have their land in any but the most fertile condition; it is, therefore, money exceptionally well invested to go to the additional expense involved in the necessary tiling. Tiling in such lands should be done with the same careful attention to the slope and to the nature of the soil and subsoil as is necessary in getting the desired results in any other case of underdrainage, as the problems to be encountered are similar. A very large percentage of the bottom land reclaimed by means of pumps is now very completely tile drained. Nearly all the small surface ditches have been replaced with tile. On some of the districts even the larger lateral ditches have been replaced with large tile, and the plans for the drainage of new districts now call for tile drains where formerly open ditches would undoubtedly have been recommended. On the Langelier District, near Havana, Ill., practically all the open ditches, both large and small, have been replaced with tile drains, until only about 2,000 feet of open ditch now remain. Thus far the system has been satisfactory, although it has not yet experienced an extremely wet season. Future experience on this district will furnish valuable data for guidance in the design of systems which are to include tile drains. The replacing of open ditches with tile drains will reduce the maintenance charges on the interior drainage channels and will effect a saving in land. The question of laying tile drains in the bottoms of broad, shallow depressions to be used as floodways for storm run-off deserves careful consideration in planning drainage improvements of this character. By laying tile drains a few feet below the bottoms of open ditches which are no longer deep enough to give a sufficient depth of drainage the combination of ditch and underdrainage may be very cheaply attained.

On account of the surface erosion and consequent sedimentation in the main ditches small open surface field ditches or laterals emptying into the main ditches are highly objectionable unless for some distance back from the junction they are dug as deep as the bottom of the main ditch. The difficulty could be avoided by the use of some sort of permanent and stable masonry inlet; otherwise small ditches should be replaced by tile.

GRAVITY SLUICEWAYS.

In the case of districts where the river falls low enough during the latter part of the summer to afford an outlet for gravity drainage, sluiceways, so arranged that they may be opened when desired, should be built under the levee near the pumping plant. The sluices should be composed of one or more parallel pipes laid nearly level and provided with suitable valves. The length of the pipes will naturally be 100 feet or more. Metal, concrete, or vitrified sewer pipe may be used in their construction. Both ends should be protected with strong concrete bulkheads, and cut-off walls should be placed around the pipes at intervals along their length to prevent seepage along their exterior surface. Cracking and failure of these end bulkheads, one of the commonest accidents in drainage districts, show that they must be made with special care, amply deep, and of liberal dimensions. If the outer end of the sluiceway is not below low-water line of the river, the bottom of the channel through which the water will flow beyond the end of the pipe should be protected from washing out by riprap, paving, or a concrete floor carried to a safe distance. Where feasible, this protection should extend below low-water line.

The sluiceway should be sufficient in size to discharge in 24 hours such an amount of water as would cover the whole drainage area one-fourth inch deep. In making this calculation the velocity of the water through the sluiceway should not be assumed to be greater than 5 feet per second. The sluiceway openings should be low enough so that their highest point will be at about the line at which it is desired to maintain the water level within the district. Their interior surfaces should be as smooth as possible in order to reduce the friction which retards the flowing water. If the entrance to the pipes is made well rounded, their maximum discharging capacity will be considerably increased. Automatic outward-opening flap valves may be put on the outer end of the pipes, but in important cases there should be also a place where the pipe may be closed by stop planks or by a gate valve if the automatic valve should get out of order.[1] On large sluices hand-operated gates of standard construction are much preferable to the automatic type, as they are positive in their action and not likely to get out of order. An excellent sluice with four compartments and four gates is shown in figure 1 of Plate III.

DESIGN OF PUMPING PLANT.

NECESSARY CAPACITY OF PUMPING MACHINERY.

Before discussing in detail the conditions determining the size needed for a pumping plant it will be illuminating to describe the general method of operation of such a plant in the latitude of Illinois. In the late summer and fall, since the precipitation is slight and is more than balanced by evaporation, the pumps are not run. During the winter storms are more frequent and evaporation is small, so that gradually the ground becomes completely saturated and the ditches full. Some districts pump occasionally throughout the whole winter; others start the pumps at some date in the spring and operate them continuously night and day for several days to remove the large accumulation of water stored up during the winter months. Gradually the water level in the ditches is lowered, and when the desired minimum is reached the machinery is stopped during the night. The water will rise in the ditches perhaps 2 or 3 feet during the night. When the pumps start up in the morning the water level at the pumping plant quickly drops a foot or more and then more slowly for the rest of the day. At points some distance away the effect of starting the pump is less quickly seen in the morning, but the lowering of the water is gradual throughout the day. After a few days of such operation the supply of water diminishes and the pumps are not run the whole day.

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[1] U. S. Dept. Agr., Office Expt. Stas. Bul. 158, Separate 9, Report of Drainage Investigations for 1904.

Later, since it is troublesome to start up a big plant for only a few hours of operation, the speed of the engine and pump is reduced to diminish its discharge, or, if there is more than one pump in the plant, only one may be run. At last, days are skipped and pumping ceases entirely. After every rain of any importance, the pumps are started and a similar cycle of operations transpires, extending over a longer or shorter period, according to the amount of water to be removed. In some districts pumping for the season ends in June, and drainage is by gravity for the remainder of the summer. In others no gravity drainage is available, and the pumps must be run later in the season. Thus it is clear that, though the pumps must constantly be in readiness for use, their operation is very intermittent. The total time of running in a whole year never exceeds 60 to 90 days of 24 hours each. In some seasons it is not more than 15 to 20 such days.

The size of pumping plant required for any particular district depends upon a variety of conditions, among which are the size and slope of the drainage district and of any other area from which drainage water is received; the amount of storage capacity available in ditches and reservoirs; the system of interior drains used and the method of construction adopted for the levee; the nature of the soil and subsoil; the method of operation of the pumping plant; the kind of crops raised and the degree of drainage required; and the amount and distribution of the rainfall throughout the year.

In the vicinity of Illinois the weight to be given to some of these factors is very small, and in the present state of our knowledge no definite allowance can be made for them. Yet, notwithstanding our ignorance concerning some of them, it may be profitable to discuss briefly the nature of the effect of each. This is a field in which much careful study is still needed and must be carried out to secure the complete knowledge necessary for the most satisfactory design and management of the pumping machinery of drainage districts.

Along the Illinois and upper Mississippi Rivers the majority of pumping plants already installed have been planned to have a maximum pumping capacity sufficient to remove in 24 hours one-fourth inch in depth of water over the entire district. This is equivalent to 6.74 cubic feet per second per square mile of drainage area, or 0.0105 cubic feet per second per acre.

It is doubtful whether any change should be made in the one-quarter inch coefficient solely on account of change in the size of the district; however, where some portions of the district are materially higher than others, the coefficient might be increased to prevent danger to the lower portions from the run-off from the higher areas. Where drainage from higher land flows into the district a larger run-off coefficient must be used for the higher area. The proper amount depends upon the nature of the country. For rolling pasture land free from undergrowth, or for cultivated corn land, probably an increase of 50 per cent in the coefficient, or a total of three-eighths inch, for such upland areas is none too great. Where feasible, the drainage from such higher areas should be diverted around the district, as previously explained, instead of being brought to the pumping plant.

With a considerable reservoir storage area within the district, the size of the pumping plant may be materially reduced. If the storage area equals 5 per cent of the total area, a 2-inch rainfall, all running into the reservoir, would raise its level only 40 inches. Therefore, if a rise of 3 feet in the reservoir can be permitted it would seem that the capacity of the pumping plant could certainly be reduced by one-third and possibly by one-half of what would otherwise be required.

If tile drainage at a depth of 3 feet or more is extensively used, the demand upon the pumping plant will be somewhat reduced, for the tiled land will act like a reservoir in storing a large amount of water and will deliver it only slowly to the ditches. On the other hand, an extensive system of shallow open field ditches will deliver the rainfall rapidly and may thus damage any especially low areas unless the capacity of the pumping plant is increased. If the levees have been properly constructed, the amount of seepage through them can never be sufficient to affect the pumping plant. Cases have been known, however, of levees so poorly built that the quantity of water coming through them was sufficient not only to endanger the stability and safety of the levees themselves, but also to increase appreciably the amount of water to be pumped.

If the soil is very stiff and tenacious the surface run-off after a heavy summer shower will be greater than from a porous open soil. During a time of flood in the river, a sandy subsoil may develop springs and boils, not only in the bottoms of the ditches, but sometimes even in the fields. The quantity of water that may be received in this way is very uncertain, but it is not probable that it is often sufficient in amount to affect the pumping plant.

A pumping plant that is designed to operate night and day may, of course, be smaller than one intended to operate only in the daytime. Practically all plants are expected to run nights occasionally, when needed. Pumping plants are operated chiefly during the months from March to June. One of the times of longest continuous operation during the whole year is always at the beginning of pumping in the spring, because if there has been no pumping during the fall and winter the rains and snows will have completely saturated the ground and filled the ditches. Hence, those districts which have had the longest experience in pumping are unanimously of the opinion that even though at that time there is no danger of immediate damage to crops, still it is best to pump occasionally during the winter so as to maintain the ground water constantly at a low level, and thus secure the great advantage of early drying of the soil in the spring and the opportunity for early cultivation and planting. Moreover, the lower the normal ground-water level is kept below the surface, the less will be the flooding effect of a severe storm.

It should be clear from the foregoing discussion that a district which gives especial care and attention to its pumping, running throughout the winter as needed and frequently during the growing season, so as to maintain a minimum ground-water level while there is any danger from the occurrence of heavy and sudden rains, will require a smaller pumping plant than would otherwise be needed.

From a surface covered with a growing hay or grain crop the run-off is less rapid than from land in corn. For corn and hay the ground water must be kept at a lower level than is necessary for grain. Corn is considered the most profitable crop in the region under consideration, and where the soil can be kept sufficiently dry it is raised as frequently as is consistent with a proper crop rotation necessary for maintaining the fertility of the soil. Much land can be observed, however, which is not sufficiently drained for the most successful raising of corn and which therefore is devoted to wheat and oats instead.

The amount and distribution of the rainfall are the chief factors determining the size of pumping plant required in a given location, and we shall proceed to a detailed discussion of all the evidence that it has been possible to gather bearing upon this point. From a consideration of all the data collected to date, the conclusion seems justified that the run-off coefficient of 0.3 inch in depth over the whole drainage district in 24 hours is the correct one to use in designing pumping plants on the Illinois and upper Mississippi Rivers. Direct observations on numerous plants in actual operation for a series of years will be needed to settle the question finally.

In the region just referred to, though some pumping on an inadequate scale has been carried on for about 15 years, all the older districts had until comparatively recently experienced trouble with their ditches and levees which prevented their pumping experience from furnishing conclusive data; from no plant has it been possible to obtain definite records going back more than a few years. Now, though there are more than 20 large plants in operating condition, it is surprising to learn that the majority of these still neglect to keep even the simplest, most fundamental records, such as would determine whether the plant is operating economically or wastefully—records which would be of great value to the district immediately concerned as well as to others.

As indirect evidence it may be stated that for many years in central Illinois the one-fourth-inch coefficient has been used in designing ditches for the drainage by gravity of flat lands and has been found adequate and satisfactory. For a much longer time in Holland and in the fens of England the coefficient of one-fourth inch or a little more has been the basis used in the designing of pumping plants. Although their total annual rainfall is about the same as in Illinois, their climate is otherwise quite different. In England and Holland there are many more rainy days, so that the rainfall is much more evenly distributed than in this country. In these countries, too, there is much less sunshine and hot weather in summer, and therefore less rapid evaporation. On the other hand, they do not have such violent and excessive rains falling during brief periods as we have.

Valuable data were obtained from the Coal Creek, Pekin-La Marsh, and East Peoria Districts on the Illinois River and from the Louisa-Des Moines District on the Mississippi River. While the records at the plants were carefully kept, unfortunately the periods covered by the most accurate portions of the records do not include any very heavy precipitation. The data obtained from each plant will be discussed in the order the plants are mentioned.

The Coal Creek Levee and Drainage District, near Beardstown, Ill., has been securing drainage by pumping since about 1898, though entire success had not been attained previous to 1909. Up until the end of 1913 the equipment of this plant was as follows: One 24-inch centrifugal pump with a nominal rated capacity of 33.5 cubic feet per second; and two 15-inch pumps, each rated at 15.5 cubic feet per second, or a total capacity of 64.5 cubic feet per second. Since the area of the watershed of the district is 7,425 acres, this would make the approximate capacity of the plant 0.21 inch in depth in 24 hours.

Since July, 1909, accurate and detailed daily records of the operation of this plant have been kept by the owners. With the knowledge gained from these records of the stages of the water in the ditches, the hours of pumping, the rainfall, and the capacities of the ditches at the various ditch stages, the capacity of the plant as operated was estimated. It was found to be a little over 0.14 inch per 24 hours. The rainfall is measured at the pumping station by means of a rain gauge of the Weather Bureau pattern. Between July 1, 1909, and May, 1910, two very heavy periods of precipitation occurred. On July 7, 8, and 9, 1909, the total rainfall was 2.8 inches, necessitating that all the pumps be run continuously for 57 hours, after which one of the smaller pumps was stopped and the other two were operated for 12 hours. The approximate run-off would be estimated for the first three days of pumping as a total of 0.38 inch. May, 1910, was a particularly wet month at this station; the total precipitation was 5.8 inches, though there were no single remarkably heavy showers. On the first two days of the month 1.4 inches fell; on the 7th, 1.4 inches; on the 11th, 1.2 inches, and from the 15th to the 17th 0.8 inch. To meet this steady wet period all three pumps were run continuously for 72 hours from the 13th to the 15th, and again for 42 hours on the 18th and 19th. The estimated amount pumped in each period would be 0.42 inch for the first and 0.35 inch for the second. During this month the plant was operated at full capacity for 70 per cent of the time, removing a total estimated depth of 3.04 inches of water from the district. In the period from July 1, 1909, to July 1, 1910, the estimated total amount pumped was 20 inches.

During the latter part of 1913 the above-described plant was replaced with a new one, equipped with three 20-inch centrifugal pumps having a combined capacity of 87.5 cubic feet per second, or a depth of 0.28 inch on the entire district per 24 hours. After installation these pumps were carefully rated at the various heads and accurate records were kept for the year 1914. The year was unusually dry, and only one storm period of any consequence occurred during the entire year. The rainfall and amount pumped for this storm period are shown in Table 1.

Table 1.—*Daily rainfall and run-off, Coal Creek Levee and Drainage District, March 25–April 11, 1914.*

Day.	Rainfall.	Run-off.	Day.	Rainfall.	Run-off.	Day.	Rainfall.	Run-off.
	Inch.	*Inch.*		*Inch.*	*Inch.*		*Inches.*	*Inches.*
Mar. 25	0.25	0.06	Apr. 1	0.32	0.10	Apr. 8	0.00	0.15
26	.00	.05	2	.00	.12	9	.00	.12
27	.75	.07	3	.00	.12	10	.00	.11
28	.25	.10	4	.00	.12	11	.00	.10
29	1.00	.24	5	.00	.10			
30	.00	.14	6	.00	.10	Total	3.57	2.10
31	.00	.16	7	1.00	.14			

Data for the above district for the year 1914 are shown in Table 2.

Table 2.—*Monthly rainfall and run-off, Coal Creek Levee and Drainage District, 1914.*

	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
	In.	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*
Rainfall	3.11	2.50	2.25	1.70	1.88	2.00	2.01	2.41	3.88	1.89	0.00	1.87	25.50
Run-off	.62	.78	2.60	2.65	1.10	.76	.40	.12	.48	.47	.49	.52	10.99

During the history of its pumping, extending over about 25 years, the Pekin-La Marsh Levee and Drainage District, near Pekin, Ill., has had several different pumping plants, but at the end of 1914 it had the following equipment: One 26-inch centrifugal pump with an approximate capacity of 30 cubic feet per second and one 12-inch centrifugal pump with a capacity of 8.5 cubic feet per second. The combined capacity in depth per 24 hours is now 0.28 inch over a watershed of 3,200 acres. Previous to January, 1912, only the 26-inch pump had been installed. Up to that time only a rough daily memorandum had been kept of the operation of the plant. However, some estimates were possible for pumping done during that time. In 1909 this plant started on March 10 and operated intermittently as required until July 25, during which time the district was drained satisfactorily. The total time of operation was forty-five and seven-tenths 24-hour days, making the total depth of water removed 10 inches. During this period the most severe test of the capacity of the plant was 4 days of steady operation, July 7–10, followed by 7 hours on July 11, or a continuous run of 103 hours. This was caused by a heavy rain on July 5, 6, and 7, totaling 4.1 inches. The amount pumped was approximately a depth of 0.96 inch from the whole district. In October, 1909, the pump drive was changed from gas engine to electricity; from that time until June 20, 1910, a total of 1,365 hours were run, equivalent to 56.9 days. The total depth pumped during this period was approximately 12.7 inches. During this period the plant was never run more than 12 hours in one day, and very seldom as much as this. As will be shown later, the year 1909, although not an extreme one, was rather wetter than the average, while 1910 was remarkably dry.

After January, 1912, an accurate daily record of the pumping was kept, and from these records the daily amount pumped has been calculated. While the capacity given for the large pump is only approximately correct, most of the pumping was done with the small pump, which had been carefully rated. In Table 3 is given a statement of the daily rainfall and amounts pumped for the largest storm periods in 1912 and 1913. The year 1914 was unusually dry and no storm of consequence occurred during the entire year.

TABLE 3.—*Daily rainfall and run-off, Pekin-La Marsh Levee and Drainage District, 1912 and 1913.*

Day.	Rainfall.	Run-off.	Day.	Rainfall.	Run-off.	Day.	Rainfall.	Run-off.
1912.	*Inches.*	*Inches.*	1912.	*Inches.*	*Inches.*	1913.	*Inches.*	*Inches.*
Apr. 26	0.54	0.05	June 10	0.00	0.03	Mar. 19	0.00	0.06
27	.00	.08	11	.00	.03	20	.15	.06
28	1.08	.12	12	.22	.03	21	1.22	.10
29	.42	.14	13	.14	.03	22	.00	.11
30	.00	.14	14	.05	.02	23	.98	.12
May 1	.22	.13	15	2.10	.03	24	.25	.13
2	.74	.15	16	.03	.13	25	.08	.18
3	.00	.23	17	.00	.14	26	.00	.17
4	.00	.16	18	.00	.08	27	.00	.10
5	.40	.13	19	.37	.05	28	.00	.10
6	.00	.12	20	.20	.04			
7	.00	.09	21	.00	.06			
			22	.00	.04			
			23	.00	.03			
Total	3.40	1.54	Total	3.11	.74	Total	2.68	1.13

It will be noted that the storms occurring in the early spring, when the ground is wet and the evaporation is small, have a much larger percentage of run-off than those occurring in summer. The monthly rainfall and run-off for the above district for the years 1912, 1913, and 1914 are given in Table 4.

TABLE 4.—*Monthly rainfall and run-off, Pekin-La Marsh Levee and Drainage District.*

Month.	1912		1913		1914	
	Rainfall.	Run-off.	Rainfall.	Run-off.	Rainfall.	Run-off.
	Inches.	*Inches.*	*Inches.*	*Inches.*	*Inches.*	*Inches.*
January	0.00	0.71	1.94	1.04	1.61	0.30
February	1.17	.58	2.48	1.18	1.61	.36
March	1.68	1.71	3.07	2.85	1.01	1.34
April	5.50	2.61	2.86	[1] 4.18	2.03	1.32
May	4.40	2.61	1.82	1.19	1.90	.83
June	4.42	1.24	1.59	.65	1.56	.17
July	4.07	.79	1.27	.19	1.44	.02
August	2.16	.29	2.74	.00	1.95	.00
September	4.40	.27	2.30	.00	5.09	.18
October	4.09	.68	2.52	.10	2.10	.21
November	1.76	1.28	2.25	.17	.14	.09
December	.81	.64	1.21	.26	.41	.06
Total	34.46	13.41	26.05	11.81	20.85	4.88

[1] Large run-off due to flow of water through pump from river during high stage.

The East Peoria Levee and Drainage District, on the Illinois River opposite Peoria, Ill., has been pumping since 1911. The plant is equipped with one 15-inch and one 18-inch centrifugal pump. The combined capacity of the pumps is 33.5 cubic feet per second, or a depth of 0.51 inch from the entire watershed of 1,550 acres per 24 hours. Accurate records were obtainable only for the period from September 15, 1913, to September 15, 1914. The weather was very dry and no period of pumping occurred which taxed the pumping plant seriously. The monthly totals of rainfall and run-off are given in Table 5.

TABLE 5.—*Monthly rainfall and run-off, East Peoria Levee and Drainage District.*

	1913.				1914.									Total.
	Sept.	Oct.	Nov.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	
	In.	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*
Rainfall	2.38	2.81	2.77	0.73	0.21	1.36	1.60	2.10	2.29	1.95	0.85	2.40	3.90	25.35
Run-off	.04	.19	.33	.38	.65	.60	1.06	1.34	1.10	.64	.35	.19	.13	7.00

The largest amount pumped on any one day during the above period was a depth of 0.1 inch on April 7, the first day after a rain of 0.95 inch. The data from this district for 1914 are of no value in the general determination of necessary capacity, but they will be used later in connection with the cost of pumping.

The Louisa-Des Moines Drainage District No. 4, about 20 miles north of Burlington, Iowa, on the Mississippi River, has been pumping since 1909. This district has a watershed area of 16,000 acres and a pumping plant equipped with two 50-inch pumps having a combined capacity of 225 cubic feet per second, or a depth of 0.33 inch on the entire watershed per 24 hours Accurate daily records were available for the years 1912 and 1913. The elevation of the land in this district with reference to the usual stages in the Mississippi River is such that for several months in each year gravity drainage is possible through sluice gates. The gates are usually open during the summer, when the storms of the greatest intensity and amount are likely to occur. Under the conditions that usually prevail the time of greatest pumping is during the spring months. The Mississippi and Iowa Rivers, which bound the district on two sides, are usually high at that time, while seepage is at the maximum and evaporation is near the minimum. When pumping was started in March, 1912, there was considerable snow and ice in the district; the rivers rose very nearly to the top of the levees, causing heavy seepage. The rainfall was about normal. Table 6 shows the daily rainfall and run-off for March and April, 1912; it is unlikely that the run-off from this district will much exceed these figures at any time.

Table 6.—*Daily rainfall and run-off, Louisa-Des Moines Drainage District No. 4, 1912.*

Day.	Rainfall.	Run-off.	Day.	Rainfall.	Runoff.	Day.	Rainfall.	Run-off.
	Inches.	*Inches.*		*Inches.*	*Inches.*		*Inches.*	*Inches.*
Mar. 26	0.84	0.18	Apr. 13		0.26	May 1	1.86	0.16
27		.21	14		.25	2		.16
28		.21	15		.25	3		.17
29		.21	16		.24	4		.18
30		.23	17	1.93	.19	5		.16
31		.23	18		.24	6		.17
Apr. 1		.22	19		.26	7		.15
2		.21	20		.17	8		.15
3		.21	21		.19	9		.15
4		.22	22		.25	10		.13
5		.22	23		.25	11		.13
6	.20	.22	24		.18	12	.27	.12
7		.23	25		.17	13		.13
8		.23	26		.17	14		.13
9		.24	27		.18	15		.00
10		.24	28		.17	16	.46	.12
11		.24	29		.17	17		.13
12	.60	.25	30		.16	18		.11

The plant was not operated an entire day, i. e., 24 consecutive hours, during this time. The object was not to remove all the water possible in one day, but to hold the stage of water in the internal drainage channels below the danger point. If the plant had been operated at full capacity for several entire days, it would have meant the skipping of a day to allow sufficient water to collect for pumping. The mean lift would also have been increased. In this case, contrary to the usual condition on pumping districts, the amount pumped each day must very closely approximate the amount of water that actually ran into the main drainage channels on that day. A study of the stage of water in the main ditch bears out this conclusion. During the pumping season of 1913 neither heavy precipitation nor high river stages occurred and the amount pumped was never more than 0.16 inch in any one day. In Table 7 are given the monthly totals of rainfall and amounts pumped.

TABLE 7.—*Monthly rainfall and run-off, Louisa-Des Moines Drainage District No. 4.*

Month.	1912		1913		Month.	1912		1913	
	Rain-fall.	Run-off.	Rain-fall.	Run-off.		Rain-fall.	Run-off.	Rain-fall.	Run-off.
	Inches.	*Inches.*	*Inches.*	*Inches.*		*Inches.*	*Inches.*	*Inches.*	*Inches.*
January	0.59	0.00	1.55	(1)	August	1.99	0.56	3.29	(1)
February	1.24	.00	2.06	(1)	September	1.86	.67	1.92	(1)
March	2.32	1.27	3.03	2.01	October	5.10	(1)	2.92	(1)
April	2.73	6.48	2.55	4.05	November	.99	(1)	.97	(1)
May	3.49	3.71	5.35	2.58	December	1.19	(1)	.61	(1)
June	1.19	2.07	3.91	2.53					
July	2.98	.40	.00	.95	Total	25.67	15.16	28.16	12.12

(1) Drainage obtained by use of sluice gates.

Tables 8, 9, and 10 show the monthly and yearly rainfall at Muscatine, Peoria, and St. Louis. These are chosen, as they are located at the corners of a triangle which includes nearly all the area in which pumping is required.

TABLE 8.—*Monthly precipitation, in inches, Muscatine, Iowa.*

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1846	2.80	4.50	2.10	5.40	3.40	4.20	1.30	0.50	5.50	1.30	1.80	1.75	34.55
1847	.79	1.11	2.94	3.30	3.50	4.60	1.20	3.30	2.10	1.21	3.45	1.00	28.50
1848	1.20	1.61	2.31	.70	3.40	2.50	5.70	9.10	3.00	4.30	1.85	3.95	39.62
1849	3.15	1.03	2.51	4.70	4.70	12.20	1.40	12.20	5.00	4.80	6.60	.87	59.16
1850	4.62	.80	2.08	3.32	3.70	3.50	5.00	13.00	3.90	2.70	3.59	2.87	49.08
1851	1.55	5.34	3.03	3.60	**12.60**	**14.30**	8.60	**14.00**	3.50	1.40	3.63	2.95	**74.50**
1852	2.52	1.00	8.60	5.30	6.50	2.20	3.70	2.80	8.30	**7.60**	5.76	5.11	59.39
1853	.40	.90	.90	**11.80**	4.60	6.40	6.60	1.70	6.20	.20	4.90	.32	44.92
1854	.40	1.80	1.23	1.76	6.21	.66	2.22	3.33	1.13	4.22	.19	.51	23.66
1855	3.25	.71	1.87	2.55	1.94	4.75	2.35	3.51	1.84	2.81	2.23	3.32	31.13
1856	1.22	5.54	.61	3.44	4.39	2.68	2.74	1.36	2.45	5.21	4.35	**7.95**	41.94
1857	.61	**5.80**	3.34	1.90	2.75	.90	4.67	6.60	1.88	1.95	2.92	1.53	34.85
1858	1.60	3.80	2.20	5.87	8.40	6.67	7.30	4.12	6.10	4.95	4.54	2.90	58.45
1859	.44	3.89	5.01	3.70	7.49	5.82	2.93	1.70	1.80	.85	1.33	1.00	35.96
1860	1.17	.43	.55	1.67	1.42	3.66	4.03	2.30	2.76	1.00	2.09	4.02	25.10
1861	1.50	2.87	4.50	3.93	3.06	1.72	2.65	2.80	9.30	7.20	2.60	2.10	44.23
1862	4.00	2.50	**9.20**	7.00	2.65	6.74	3.45	7.25	4.50	2.85	1.32	1.70	53.16
1863	1.70	1.50	2.00	1.52	1.89	.91	.25	4.15	2.41	3.54	1.44	5.52	26.83
1864	1.05	.25	2.59	3.43	3.39	5.75	3.25	2.31	3.20	3.11	2.59	1.85	32.77
1865	.46	2.90	2.76	6.02	1.05	3.69	4.50	4.25	4.23	3.53	.20	.62	34.21
1866	3.38	.58	1.94	3.91	1.18	2.82	5.18	3.36	4.71	1.94	1.01	2.85	32.86
1867	.75	3.62	1.93	2.34	5.77	4.70	3.24	1.65	3.44	1.00	2.85	.95	32.24
1868	.30	.95	5.55	7.43	7.07	2.50	4.59	2.85	5.18	.94	2.71	.84	40.91
1869	1.62	1.63	1.53	2.64	4.47	9.15	8.55	5.35	1.95	1.59	2.47	2.41	43.30
1870	2.04	.18	3.37	.38	1.86	.93	1.05	4.46	4.59	3.95	.87	.93	24.61
1871	2.15	2.09	1.88	1.95	1.93	6.71	3.28	6.12	1.07	3.73	2.30	2.90	36.11
1872	.06	[1].30	2.55	3.92	7.57	5.17	2.97	5.84	4.15	.84	1.30	.74	35.41
1873	**8.49**	.35	1.90	1.45	4.31	1.91	2.61	....	1.30	1.44	1.18	3.49	28.43
1874	4.72	2.88	2.39	2.93	1.51	3.70	2.26	3.54	6.58	.90	2.49	.88	34.78
1875	.65	1.50	1.99	.90	2.65	4.97	6.72	2.56	**9.38**	1.27	.63	4.37	37.59
1876	3.23	2.85	3.54	4.00	7.38	4.45	**9.15**	5.72	6.53	1.84	3.48	1.40	53.57
1877	1.58	2.00	4.28	3.57	2.25	7.44	4.82	4.49	1.25	6.03	3.06	3.01	43.78
1878	.40	1.56	2.87	2.36	7.49	4.09	3.57	7.43	2.81	4.35	.77	1.60	39.30
1879	1.20	1.07	2.18	1.81	4.54	3.78	3.40	4.56	2.37	2.85	4.65	1.42	33.83
1880	3.02	1.63	3.62	3.02	3.25	7.22	3.84	4.48	3.17	.25	.97	1.31	35.78
1881	1.34	3.49	2.58	2.11	2.43	10.38	3.77	1.36	6.59	7.03	2.83	1.75	45.66
1882	.84	1.10	2.98	4.00	8.36	8.25	4.55	1.75	1.38	4.29	**7.15**	2.02	46.67
1883	1.66	4.67	.75	5.00	6.19	4.77	4.31	1.45	1.19	6.23	3.45	1.45	41.12
1884	1.05	1.40	4.28	2.08	5.57	4.03	3.63	5.77	5.23	6.46	1.57	4.42	45.49
1885	2.38	2.21	1.25	[1] 4.00	4.18	4.81	5.03	7.38	2.88	2.80	.83	2.32	39.07
1886	4.21	1.43	4.16	2.62	5.05	[1] 1.30	.32	2.62	3.05	4.70	.95	.71	31.12
1887	1.88	4.12	1.14	1.26	2.26	2.10	2.90	2.40	3.49	2.24	.98	3.53	28.30
1888	1.49	.70	3.10	[1] 1.60	6.78	3.82	3.39	7.24	2.00	1.50	4.43	2.84	38.89
1889	1.43	1.38	.65	4.28	4.17	5.68	6.69	1.15	3.95	1.04	1.65	1.40	33.47
1890	1.90	1.70	3.29	1.12	3.61	6.68	1.88	2.35	2.52	4.24	1.38	1.20	31.87
1891	1.75	2.19	3.50	[1] 3.10	2.51	4.87	3.29	5.20	1.35	1.49	[1] 3.50	2.51	35.26
1892	1.35	1.08	3.90	4.31	10.32	8.83	3.85	1.70	.47	.82	2.21	2.56	42.30
1893	1.35	1.60	3.49	5.33	2.35	3.61	2.59	1.02	5.05	1.14	2.40	1.27	31.20
1894	1.80	1.25	2.44	5.33	2.05	2.04	.24	2.74	5.85	2.10	2.99	.76	29.59
1895	1.83	.84	1.75	.83	3.05	.98	7.82	4.22	2.12	1.02	2.12	2.62	29.20
1896	1.10	1.18	1.32	5.72	5.21	2.02	5.59	4.43	3.63	1.89	1.09	1.27	34.45
1897	5.46	2.72	2.80	5.23	1.59	2.54	3.51	1.21	2.50	.78	2.07	1.94	32.35
1898	3.33	1.72	3.64	3.74	5.30	3.34	.73	6.22	2.34	3.74	1.76	.51	36.37

[1] Values taken from surrounding stations.

TABLE 8.—*Monthly precipitation, in inches, Muscatine, Iowa*—Continued.

Year.	Jan.	Feb.	Mar.	Apr.	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1899....	.38	1.52	1.75	3.18	7.61	2.35	2.67	3.14	1.08	1.24	1.52	2.21	28.65
1900....	1.72	2.65	3.86	2.02	4.05	1.57	3.65	6.31	4.31	1.96	1.36	.99	34.45
1901....	1.46	1.63	2.81	1.81	1.26	4.50	1.44	.40	2.17	.99	.95	1.72	21.14
1902....	.59	.95	2.33	2.14	4.66	8.79	7.93	8.94	4.21	3.28	3.28	2.41	49.51
1903....	.58	1.72	2.01	3.76	6.80	3.46	4.79	5.13	5.26	1.82	.97	.99	37.29
1904....	2.27	.71	2.65	2.83	2.87	2.35	3.52	6.44	2.14	1.01	.20	2.43	29.42
1905....	.72	2.15	1.42	3.08	2.84	6.73	2.81	3.52	1.17	4.05	2.41	.87	31.77
1906....	3.12	2.91	3.12	1.88	3.76	3.89	1.96	2.75	2.78	1.42	2.55	1.69	31.83
1907....	4.78	.49	2.55	2.35	8.26	4.66	8.80	5.20	2.55	1.49	.97	.81	42.91
1908....	.66	2.03	2.17	2.34	6.10	3.86	3.72	8.77	2.02	2.10	2.80	.56	37.13
1909....	2.19	2.30	1.58	5.46	3.35	4.43	3.63	2.61	1.96	1.68	5.53	2.90	37.62
1910....	1.73	.74	.36	2.40	4.39	2.62	3.04	3.17	2.80	.66	.42	.96	23.29
1911....	1.38	3.91	.95	3.45	2.65	2.33	4.35	3.11	8.58	2.48	4.22	2.02	39.43
Mean	1.90	1.97	2.66	3.36	4.40	4.46	3.87	4.28	3.55	2.64	2.40	2.06	37.58

TABLE 9.—*Monthly precipitation, in inches, Peoria, Ill.*

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual
1855....	......	......	......	......	......	......	......	......	......	......	......	3.76	......
1856....	0.80	1.03	0.25	1.72	4.03	1.50	2.83	1.39	0.76	1.66	4.00	6.13	26.10
1857....	.37	5.33	3.84	1.39	2.80	2.77	1.40	5.61	2.16	2.01	1.33	1.50	30.51
1858....	1.48	1.95	3.28	6.25	**10.64**	5.95	5.75	3.24	2.96	3.24	4.85	3.67	**53.26**
1859....	1.50	1.42	**5.82**	2.60	3.17	2.18	.67	4.14	2.84	2.15	2.40	1.23	30.12
1860....	1.86	2.40	1.13	1.64	2.00	4.95	**8.87**	2.39	2.00	.70	3.13	3.08	34.15
1861....	1.25	2.46	3.96	4.95	2.19	2.31	2.31	2.78	3.72	2.33	1.09	.94	30.29
1862....	4.27	.70	2.71	5.03	1.46	3.67	7.74	**9.04**	5.09	1.61	1.81	5.20	48.33
1863....	2.83	3.20	2.61	1.52	2.97	.45	4.82	2.24	2.51	3.92	.71	4.49	32.27
1864....	1.42	.41	2.20	4.81	1.88	2.55	2.92	1.56	4.81	1.53	3.82	3.06	30.97
1865....	.22	4.01	3.57	4.27	2.34	1.86	5.77	3.61	8.31	1.67	.31	1.08	37.02
1866....	3.21	1.10	2.54	2.65	2.57	2.61	5.17	3.97	6.50	2.87	.51	2.05	35.75
1867....	1.36	2.88	1.74	1.57	4.40	2.92	2.65	2.26	.60	1.10	1.93	1.21	24.62
1868....	.77	.75	5.38	3.18	7.85	1.43	1.47	2.74	4.46	1.41	4.50	1.81	35.75
1869....	.99	2.62	1.71	3.59	6.09	8.35	7.35	3.39	.74	1.53	3.13	2.63	42.12
1870....	2.05	.33	4.37	.45	1.62	.75	.68	3.26	3.56	4.27	1.21	1.02	23.57
1871....	2.45	1.62	3.24	2.58	1.93	3.47	3.76	4.95	.65	3.37	2.09	2.04	32.15
1872....	.20	.69	2.50	2.95	2.38	9.76	7.80	4.54	4.13	.80	2.00	1.07	38.82
1873....	3.47	1.29	1.30	4.76	4.78	2.96	4.25	1.25	3.65	2.26	1.46	**7.15**	38.58
1874....	3.04	1.45	1.11	2.90	2.51	1.95	1.46	5.60	1.15	1.00	2.20	.67	25.04
1875....	.32	2.20	2.05	2.00	4.23	3.00	8.28	1.02	9.63	3.46	.71	2.39	39.29
1876....	2.60	2.00	4.70	2.66	3.94	6.17	5.54	3.14	4.51	4.86	2.63	.28	43.03
1877....	.92	.06	3.32	2.86	2.57	9.43	3.01	2.04	2.83	**5.68**	3.65	3.45	39.82
1878....	.50	2.25	2.10	3.75	4.45	3.49	2.58	4.42	.97	3.96	.91	2.08	31.46
1879....	1.05	.97	1.80	2.95	.93	3.23	3.42	1.88	3.72	2.17	4.93	1.92	28.97
1880....	3.38	3.95	3.30	5.94	6.73	2.32	3.17	3.38	3.09	1.75	1.92	.96	39.89
1881....	.52	3.51	3.52	1.62	3.50	7.20	2.43	1.38	4.05	5.56	4.26	3.50	41.05
1882....	1.27	3.21	3.12	2.41	6.34	**11.18**	2.91	1.92	1.53	3.76	2.08	1.76	41.49
1883....	1.31	4.14	.77	6.18	6.54	4.39	3.57	.57	2.93	3.57	4.19	1.37	39.53
1884....	.70	3.18	2.17	2.62	5.50	3.87	3.67	4.13	5.76	4.80	2.19	3.21	41.80
1885....	2.63	.87	.24	4.44	1.70	4.07	4.73	2.64	5.28	2.32	1.04	2.44	32.40
1886....	2.41	1.86	2.25	2.75	2.90	3.67	.47	3.57	4.68	1.81	1.34	.89	28.60
1887....	1.10	5.45	.94	1.53	1.24	1.53	2.85	2.72	2.53	2.14	1.62	3.65	27.30
1888....	1.87	1.66	4.03	1.18	6.72	1.84	6.48	2.30	4.79	2.29	2.67	2.39	38.22
1889....	1.70	.84	1.50	2.79	3.92	6.30	7.64	1.23	2.61	2.28	2.91	1.33	35.05
1890....	2.80	1.36	2.73	2.33	2.74	2.42	.72	2.39	2.12	3.45	1.79	.41	25.26
1891....	1.68	1.90	2.68	3.64	1.97	3.31	2.82	5.71	2.00	.71	4.08	2.39	32.89
1892....	1.25	1.84	2.45	4.54	7.70	6.05	3.08	.73	2.35	1.20	2.72	1.75	35.66
1893....	.87	2.92	3.01	**7.86**	4.65	1.82	2.48	.44	3.02	.70	2.21	1.74	31.72
1894....	2.60	1.48	3.06	2.22	3.58	4.18	1.00	2.50	4.42	1.45	2.92	1.58	30.99
1895....	1.32	.37	1.02	2.89	1.84	1.67	8.72	2.27	4.92	.67	4.17	5.86	35.72
1896....	1.30	1.95	1.05	4.47	5.74	2.23	7.02	4.69	4.86	.23	2.20	.40	36.14
1897....	**5.39**	1.19	4.70	2.87	1.29	2.11	4.65	1.02	.93	.04	3.48	1.16	28.83
1898....	4.08	2.59	5.74	3.02	5.54	3.37	.47	3.26	6.05	3.00	2.03	.93	40.08
1899....	.72	1.96	2.97	1.36	6.03	2.60	1.69	1.27	5.24	2.78	2.25	2.12	30.99
1900....	1.92	**5.64**	1.42	1.09	5.54	1.44	2.45	5.39	2.94	2.90	1.87	.39	32.99
1901....	1.98	1.24	4.31	.81	1.50	4.32	3.97	1.29	2.64	.90	.80	2.26	26.02
1902....	.67	1.41	2.71	2.29	2.99	9.60	7.30	7.42	6.78	3.78	2.83	1.54	49.32
1903....	.89	1.70	3.66	5.15	4.22	2.39	4.91	7.22	5.78	2.13	.85	.95	39.85
1904....	1.87	1.29	4.42	3.48	4.06	2.44	5.58	4.13	6.67	.10	.12	1.33	35.49
1905....	1.15	1.45	2.00	3.99	4.53	5.13	4.24	1.36	1.78	2.77	2.45	1.60	32.45
1906....	1.70	1.85	2.65	2.77	2.88	3.24	2.48	1.59	4.92	1.00	2.42	1.65	29.05
1907....	**5.39**	.14	2.34	2.82	2.08	3.99	4.89	6.60	2.94	.35	1.68	1.66	34.88
1908....	.59	3.98	2.50	4.08	7.76	4.09	3.94	2.78	.82	.71	1.89	.82	33.96
1909....	1.55	3.67	1.81	7.17	3.72	3.56	4.57	.79	3.68	3.59	5.53	2.50	42.14
1910....	1.97	1.10	.52	3.56	4.49	.78	3.23	.68	3.12	1.69	.79	1.25	23.18
1911....	2.37	2.39	2.64	2.69	1.03	6.64	2.58	1.73	**12.30**	2.65	3.01	2.13	42.16
Mean	1.77	2.06	2.66	3.21	3.83	3.77	3.95	3.03	3.75	2.26	2.35	2.14	34.78

Table 10.—*Monthly precipitation, in inches, St. Louis, Mo.*

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Annual.
1830	......	......	......	......	......	......	4.97	**12.25**	5.16	5.79	1.28	0.55	......
1831	2.08	1.06	4.58	4.39	1.95	1.34	1.88	2.51	1.99	6.76	4.08	3.68	36.30
1836	......	......	......	......	......	......	3.66	6.51	5.90	2.59	4.20	5.02	......
1837	.84	1.35	3.13	2.34	3.00	3.46	2.48	2.73	2.85	.79	1.96	2.01	26.94
1838	3.72	1.11	1.51	3.36	1.68	3.73	3.13	4.47	.06	3.06	2.09	.44	28.36
1839	2.21	2.50	2.59	5.46	7.93	7.26	5.71	2.89	2.45	3.96	2.48	2.00	47.44
1840	1.80	1.38	2.10	3.31	4.58	6.27	2.36	7.15	3.96	6.30	1.73	.71	41.65
1841	.84	.88	4.99	3.85	2.38	1.67	3.09	5.63	3.22	6.81	5.44	3.93	42.73
1842	.45	3.90	2.21	3.48	3.22	5.12	1.76	2.64	2.17	2.57	2.38	2.39	32.29
1843	2.34	1.90	3.49	4.87	4.15	3.95	2.49	1.32	2.19	1.55	4.82	1.72	34.79
1844	3.36	1.73	4.84	3.86	**11.26**	6.85	8.13	.45	.30	2.25	1.17	1.61	45.81
1845	1.83	1.07	3.18	2.28	4.42	10.01	4.75	6.23	1.03	1.16	1.10	.93	37.99
1846	2.98	1.27	1.27	4.84	3.75	5.21	.84	4.73	4.84	2.71	2.11	**10.90**	45.45
1847	2.12	3.58	2.28	3.98	4.36	8.61	5.37	.90	3.26	**8.74**	**8.63**	.89	52.72
1848	1.86	2.27	6.61	3.16	8.10	**17.07**	5.37	9.74	1.12	2.41	1.91	5.74	65.36
1849	4.18	.56	2.70	2.64	2.71	6.46	9.40	5.15	5.81	2.17	2.11	1.82	45.71
1850	1.94	4.10	5.63	7.68	7.47	1.47	4.83	2.10	3.74	2.71	6.24	2.59	50.50
1851	.61	6.74	3.14	4.70	2.83	6.19	1.77	8.97	.49	1.51	1.99	3.90	42.84
1852	.99	2.12	7.67	2.28	5.19	10.25	3.36	1.60	1.47	5.26	3.29	3.48	46.96
1853	.52	1.67	.79	3.24	3.64	3.23	4.10	5.48	4.67	.96	1.51	1.08	30.89
1854	1.18	3.11	7.49	7.60	6.30	3.21	.92	1.80	1.44	4.15	1.94	1.49	40.62
1855	4.66	.70	2.89	2.65	7.46	4.27	5.17	6.53	3.89	3.89	5.16	3.10	50.37
1856	1.03	3.64	1.06	6.35	3.03	1.24	4.61	6.32	3.51	2.10	4.90	4.29	42.08
1857	.41	7.74	1.80	1.72	4.81	3.71	2.82	4.15	3.18	3.02	3.80	1.87	39.03
1858	3.42	2.12	3.96	6.07	10.64	6.69	8.03	2.87	3.86	7.73	4.92	8.52	**68.83**
1859	2.32	5.35	7.32	4.89	6.60	11.02	5.54	2.93	4.44	1.80	5.43	3.76	61.40
1860	1.80	2.60	1.16	2.03	2.29	6.58	2.97	2.96	2.11	1.58	1.63	2.08	29.79
1861	1.16	2.01	7.38	3.18	4.39	4.96	2.04	3.44	4.14	2.85	1.39	1.09	38.03
1862	4.01	.80	4.11	4.82	2.51	2.85	3.61	1.32	6.27	3.73	3.59	6.38	44.00
1863	4.11	3.99	3.02	1.55	2.68	3.16	2.51	6.93	1.56	4.76	2.15	4.03	40.45
1864	2.74	.82	1.71	5.58	3.90	.41	3.60	4.91	2.82	3.15	5.25	2.72	37.61
1865	.87	3.75	**8.61**	3.31	5.66	5.21	7.94	1.97	2.60	3.33	.00	3.63	46.88
1866	4.16	2.24	2.80	1.56	2.24	5.59	3.68	3.71	**10.53**	2.01	1.37	1.87	41.75
1867	2.28	4.81	2.37	.53	8.26	5.64	3.71	2.29	.17	1.31	2.74	3.65	37.76
1868	1.71	.55	7.66	7.08	3.96	1.58	2.03	8.53	5.25	2.11	2.04	3.09	45.59
1869	2.02	2.49	4.24	4.61	3.60	6.25	2.49	5.51	1.70	3.42	7.48	3.16	46.97
1870	2.25	.33	2.76	2.39	2.73	1.38	1.59	6.55	1.14	3.35	1.94	2.76	29.17
1871	2.53	2.92	1.27	.49	3.15	2.51	1.64	3.55	.25	2.07	1.83	1.17	23.28
1872	.64	1.15	2.43	2.77	6.04	4.28	4.59	.93	3.38	.55	2.01	1.70	30.47
1873	3.53	1.52	2.10	6.88	5.73	6.68	5.96	.07	3.02	3.27	1.64	5.10	45.50
1874	3.14	3.66	4.36	3.44	3.70	2.00	5.71	4.70	2.30	1.09	2.32	1.46	37.88
1875	.54	2.59	4.08	2.53	5.48	10.84	**9.50**	2.66	.24	1.23	.89	2.42	43.00
1876	4.75	2.86	6.90	2.25	3.13	3.43	5.90	5.03	7.63	1.66	1.74	.18	48.46
1877	1.24	.88	3.41	3.03	3.11	8.69	2.88	2.61	3.56	4.92	3.76	3.34	41.43
1878	3.36	1.69	2.79	6.74	4.63	2.40	3.92	4.75	3.42	3.27	1.38	3.48	40.83
1879	1.64	1.48	1.92	2.31	.95	4.04	1.97	2.23	1.34	.68	4.30	2.84	25.70
1880	3.83	2.65	2.51	3.31	3.44	2.56	5.17	1.53	3.10	2.09	2.67	1.80	34.66
1881	.49	4.16	1.95	3.14	3.96	2.74	2.13	.31	3.14	7.21	6.74	1.40	37.37
1882	2.80	**8.94**	3.49	3.58	4.55	4.53	3.84	2.20	1.73	2.44	3.24	1.81	43.15
1883	.94	5.88	2.29	3.31	2.89	5.04	4.31	3.34	.01	6.60	3.71	1.78	40.10
1884	.79	4.43	3.00	4.15	2.68	4.52	2.86	1.21	6.04	2.48	2.30	6.18	40.64
1885	3.26	.87	.40	4.84	2.80	7.68	2.58	2.96	8.98	7.51	1.68	2.03	45.59
1886	3.11	1.71	3.04	2.10	7.84	7.09	.55	2.44	9.60	.85	3.36	2.65	44.34
1887	.65	3.68	3.54	4.36	5.27	2.54	2.74	1.14	2.47	.76	4.61	3.54	35.30
1888	2.15	2.39	3.79	1.88	3.81	8.09	2.09	6.66	1.31	2.59	4.40	2.01	41.17
1889	3.04	4.78	1.62	1.68	3.80	4.72	2.02	.85	3.54	1.65	4.43	1.03	33.16
1890	**7.47**	2.86	5.99	4.05	5.81	3.18	.37	2.43	1.80	.86	1.55	1.32	37.69
1891	1.35	2.95	2.29	2.29	2.73	5.97	1.50	2.75	1.43	.65	5.30	1.32	30.53
1892	1.52	4.89	1.92	7.60	7.87	2.73	4.64	1.75	1.59	1.66	3.46	1.99	41.62
1893	.33	2.98	5.10	**10.84**	5.42	3.49	2.49	.65	3.69	1.66	1.36	1.32	39.33
1894	2.56	2.88	2.69	2.68	3.61	1.12	1.35	1.66	3.11	1.56	1.49	2.73	27.44
1895	1.65	.43	2.82	.46	3.16	2.46	7.26	2.08	2.01	.23	3.98	4.66	31.20
1896	1.43	2.81	2.03	2.43	9.12	4.57	4.67	2.12	2.42	1.20	3.70	1.05	37.55
1897	3.75	2.67	8.25	4.66	1.59	5.32	3.23	.66	.09	.31	6.21	3.43	40.17
1898	4.53	1.71	7.73	3.85	8.55	3.85	7.44	.87	3.23	4.34	2.07	1.03	49.20
1899	1.66	3.40	3.96	1.98	6.32	2.32	4.54	2.77	1.27	2.89	1.95	1.55	34.61
1900	.65	5.09	1.45	1.83	4.47	2.62	3.85	1.30	2.68	2.07	3.10	.40	29.51
1901	1.12	1.86	2.94	2.35	2.69	3.92	1.47	.76	.64	2.12	1.21	3.72	24.80
1902	1.18	.83	4.50	2.49	3.04	7.86	2.34	5.20	1.98	2.00	3.20	3.81	38.43
1903	1.76	3.14	3.20	2.79	2.08	5.71	2.68	6.16	3.06	1.37	.61	1.25	33.81
1904	3.15	.84	7.87	3.25	2.88	4.64	3.09	2.62	2.97	.50	.54	1.36	33.71
1905	2.47	1.12	2.35	2.32	4.67	2.72	4.42	2.58	5.56	6.64	1.63	2.06	38.54
1906	3.57	2.92	4.53	1.98	2.61	2.80	.98	3.72	4.40	1.25	4.67	2.09	35.52
1907	7.35	1.12	2.39	3.65	5.57	4.96	3.32	4.36	1.57	3.15	1.89	2.06	41.39
1908	2.08	3.39	3.43	3.84	7.72	3.02	4.24	1.55	1.24	.21	2.83	.64	34.19
1909	3.20	3.94	3.69	6.18	5.99	2.63	7.34	.66	4.22	3.40	4.36	1.89	47.50
1910	2.73	3.22	.14	4.09	5.23	4.24	4.21	1.90	6.09	3.98	.30	1.18	37.31
1911	.85	3.02	2.22	7.46	2.26	1.34	.64	3.51	7.09	2.63	3.44	1.67	36.13
Mean	2.27	2.67	3.55	3.70	4.53	4.74	3.70	3.42	3.11	2.85	2.85	2.58	39.83

In each of Tables 8, 9, and 10 the maximum for each month is shown by heavy-faced figures, as is also the yearly maximum. It will be noted that the greatest mean monthly rainfall is in June in two cases and in July in the other, thus showing that the maximum rainfall is to be expected during the months when farming operations are at their height. The mean annual precipitation for the three stations is 37.39 inches. A study of the tables has shown that a monthly rainfall of 6 inches occurs a little oftener than once each year. A monthly rainfall of 8 inches occurs once in 3.5 years, and one of 10 inches once in 11 years. A rainfall of 15 inches during a period of two successive months occurs once in 3.6 years. It would appear from these figures that a pumping plant should be able to handle a monthly rainfall of at least 8 inches.

Table 11 shows the average number of storms per year, of the given intensities, for short periods of from 1 to 10 days, that may be expected in the section under consideration. This table was obtained from a study of the daily precipitation records of Muscatine, Peoria, and St. Louis.

A study of the rainfall records of Muscatine, Peoria, and St. Louis shows that about two-thirds of the storms of the various intensities given in Table 11 occurred from April to September, inclusive, the months during which crops are most injured by occasional flooding. During the six months from October to March, inclusive, occasional flooding of the land would do little if any damage in the section under consideration. Therefore the storms occurring during the winter months may be disregarded in the determination of the proper capacity of pumping plant. This can be done by reducing the number of storms of each given intensity by one-third. For instance, Table 11 shows that on the average a storm of 2 inches in two days occurs three times a year. Only two of these storms occur at a time of the year when damage to crops would be caused by flooding.

TABLE 11.—*Average yearly number of storm periods of from 1 to 10 days' duration.*

[Based upon daily precipitation records of the United States Weather Bureau for Muscatine, Iowa, 1854–1914; Peoria, Ill., 1856–1914; and St. Louis, Mo., 1843–1914.]

Total rainfall.	Average number of storms per year.									
	1-day period.	2-day period.	3-day period.	4-day period.	5-day period.	6-day period.	7-day period.	8-day period.	9-day period.	10-day period.
2 inches	2.018	3.059	3.615	4.071						
3 inches	.479	.900	1.172	1.456	1.580	1.722				
4 inches	.172	.320	.438	.545	.651	.775	0.847	0.912		
5 inches	.041	.083	.142	.160	.195	.290	.355	.444	0.479	0.520
6 inches	.012	.024	.041	.047	.071	.077	.100	.118	.165	.207
7 inches		.006	.012	.018	.030	.047	.053	.059	.071	.077
8 inches					.006	.006	.012	.024	.024	.030
9 inches						.006	.012	.018	.018	.018
10 inches								.006	.006	.006

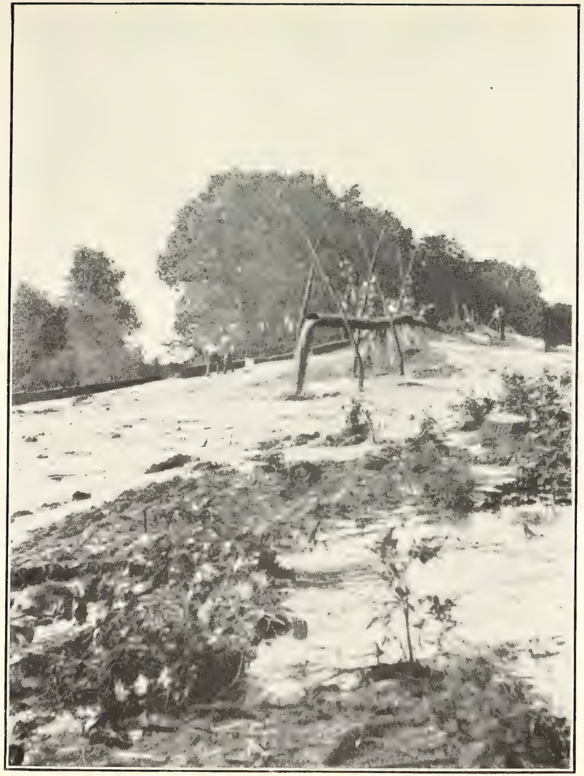


FIG. 1.—HYDRAULIC FILL LEVEE UNDER CONSTRUCTION. PIPE DISCHARGING SAND AND WATER.

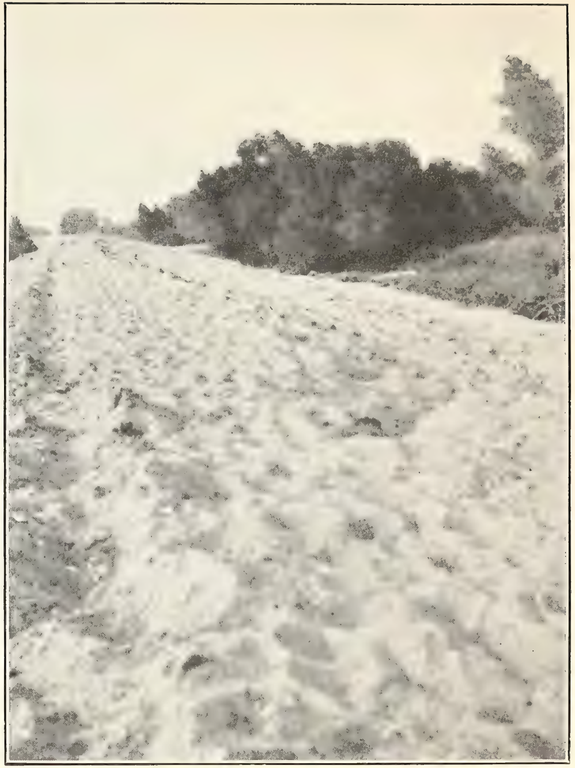
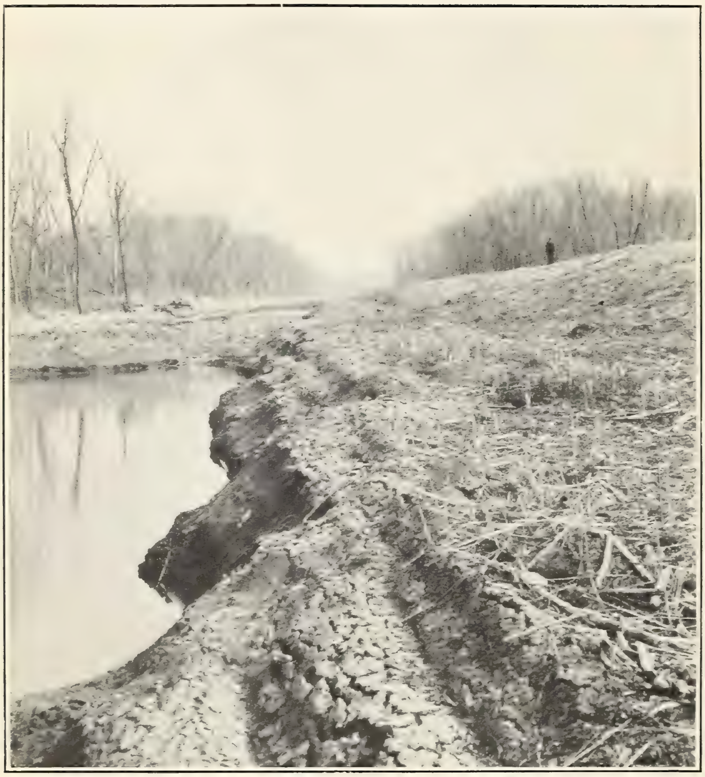


FIG. 2.—FINISHED HYDRAULIC FILL SAND LEVEE.



LEVEE CONSTRUCTED BY DRAG LINE EXCAVATOR, MUSCATINE-LOUISA LEVEE AND DRAINAGE DISTRICT.

Note earth barriers across borrow pit.

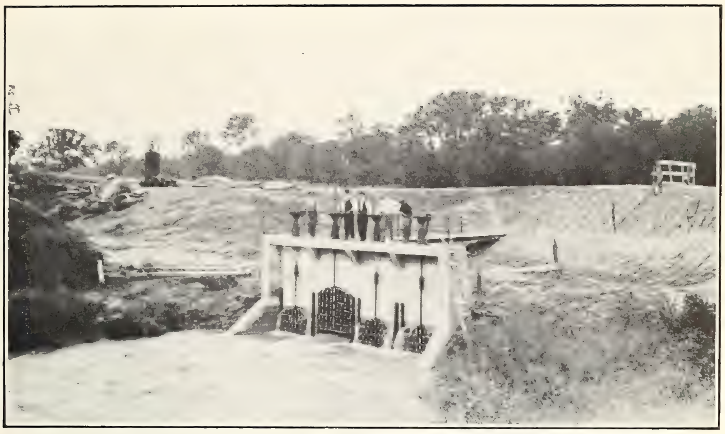


FIG. 1.—SLUICE GATE IN OPERATION, ONE GATE OPEN, DES MOINES COUNTY DRAINAGE DISTRICT NO. 1.

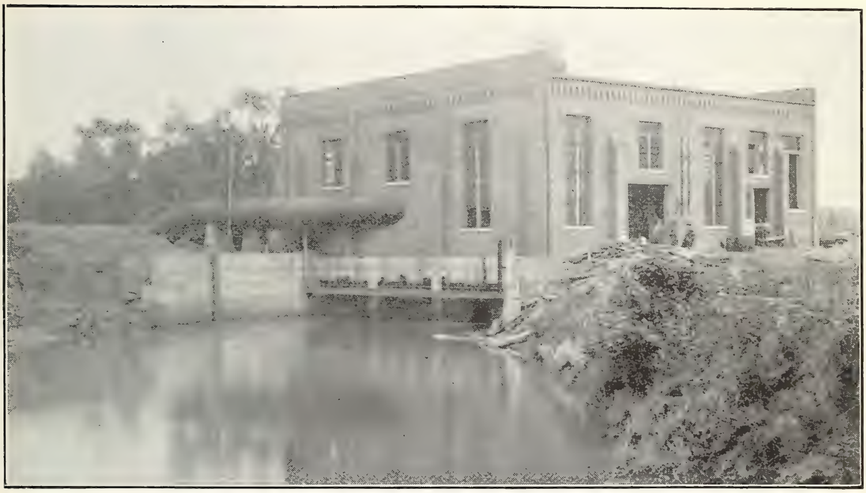


FIG. 2.—SUCTION SIDE OF LAZWELL PUMPING PLANT OF DES MOINES COUNTY DRAINAGE DISTRICT NO. 1.

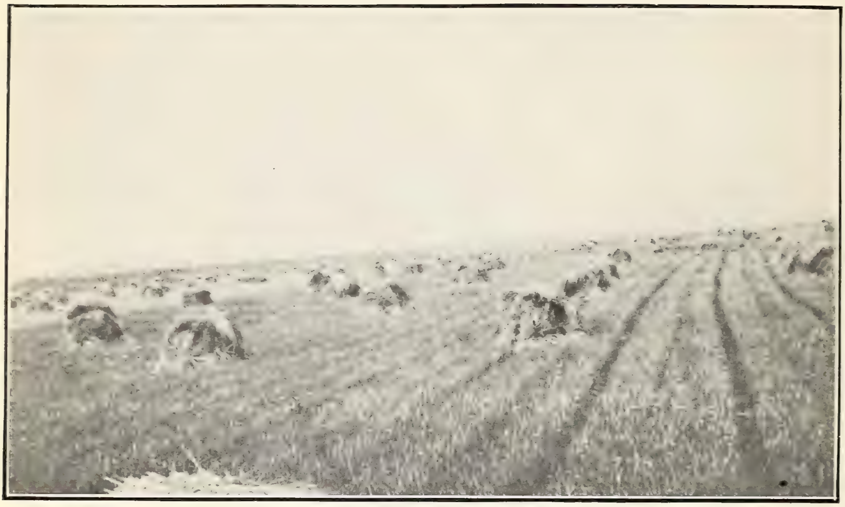


FIG. 1.—CROP OF 1910, COAL CREEK LEVEE AND DRAINAGE DISTRICT.

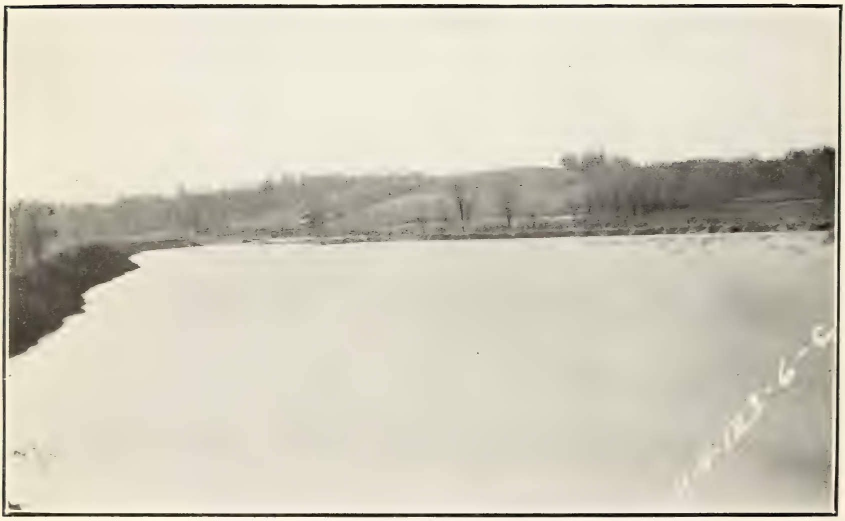


FIG. 2.—HOOD BASIN, COAL CREEK LEVEE AND DRAINAGE DISTRICT.

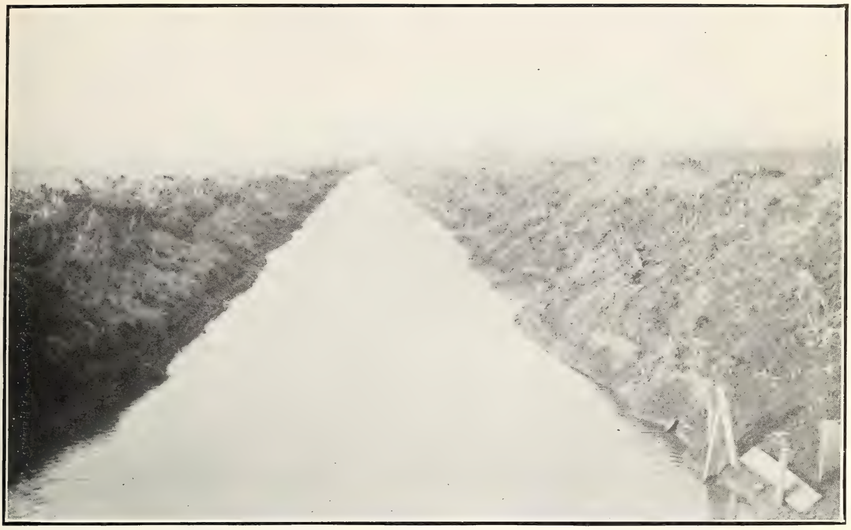


FIG. 1.—MAIN DITCH, COAL CREEK LEVEE AND DRAINAGE DISTRICT.

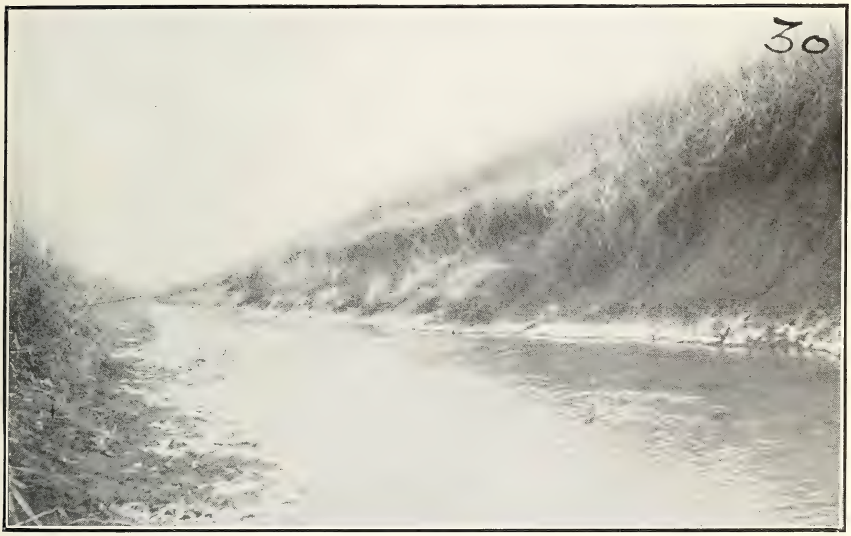


FIG. 2.—BRANCH A, COAL CREEK LEVEE AND DRAINAGE DISTRICT.

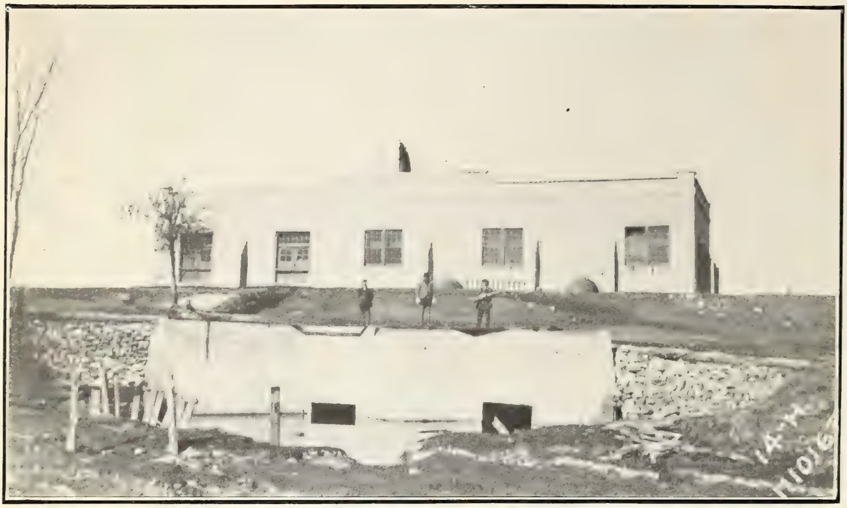


FIG. 1.—PUMPING PLANT AND DISCHARGE BAY, LOUISA-DES MOINES DRAINAGE DISTRICT.

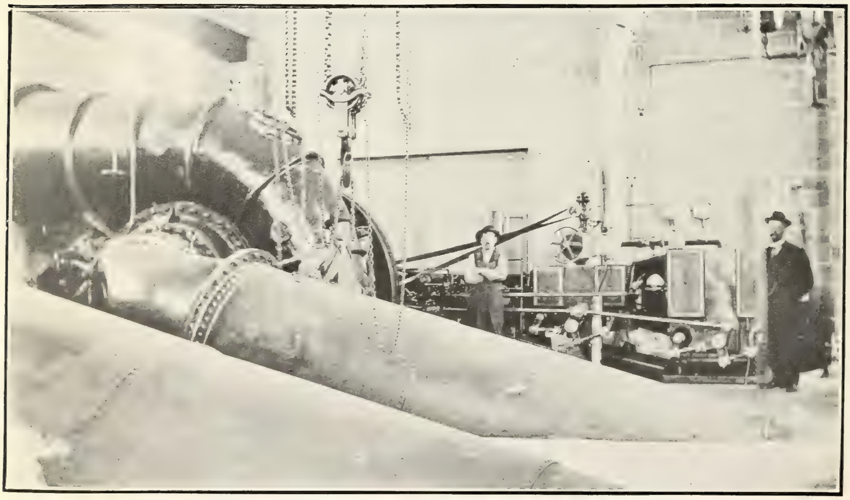
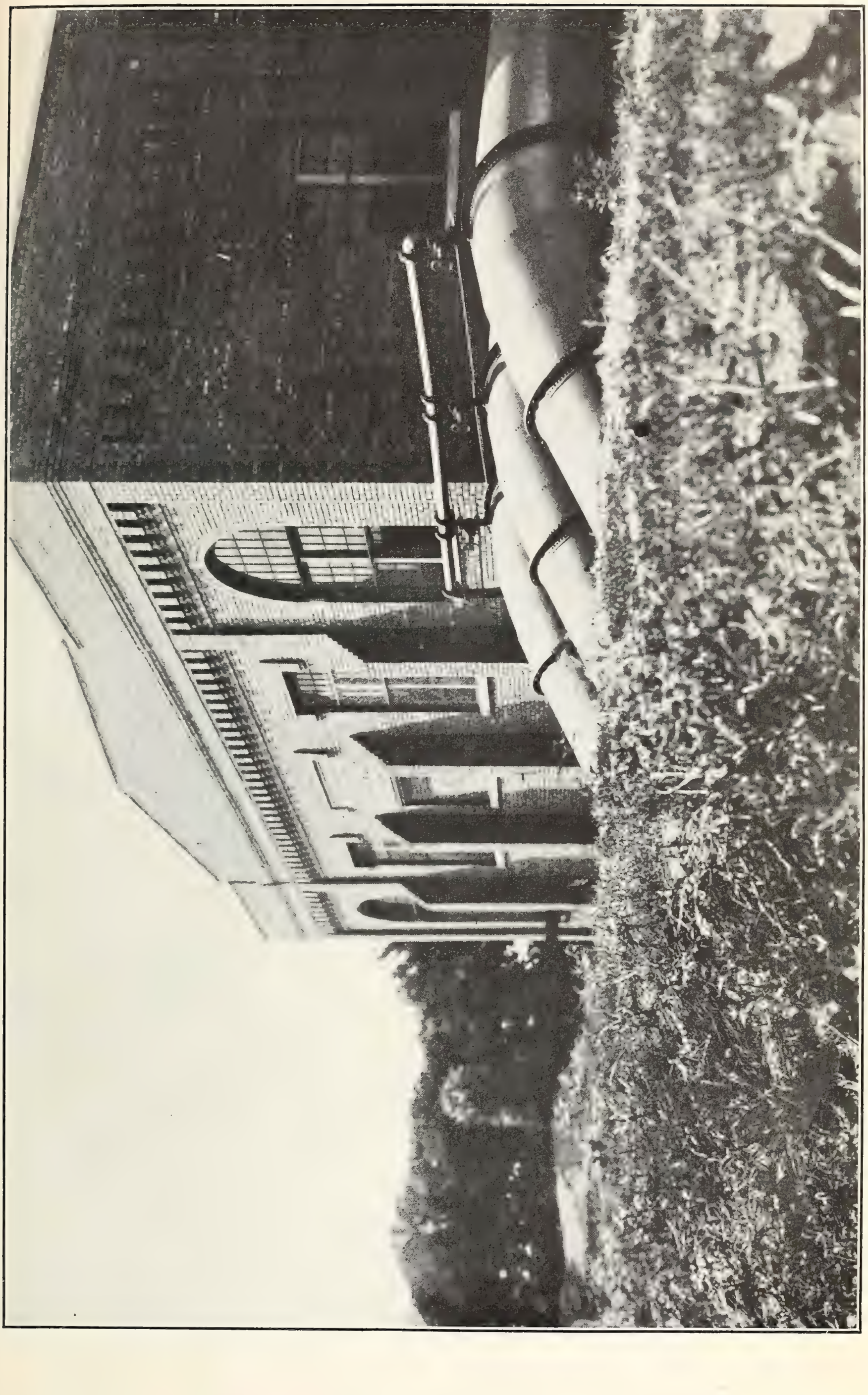


FIG. 2.—ENGINE ROOM, LOUISA-DES MOINES DRAINAGE DISTRICT.



VIEW OF DISCHARGE PIPES OF LAZWELL PUMPING PLANT OF DES MOINES COUNTY DRAINAGE DISTRICT NO. 1.

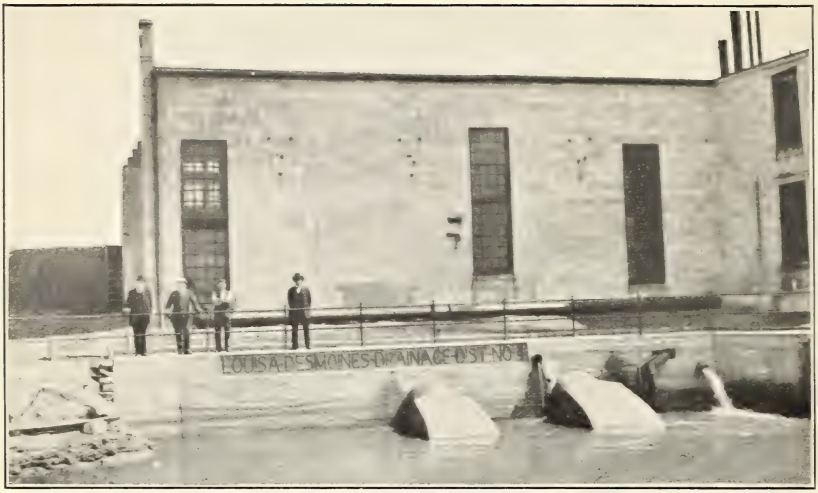


FIG. 1.—REAR OF PUMPING PLANT, LOUISA-DES MOINES DRAINAGE DISTRICT, SHOWING SUCTION PIT.

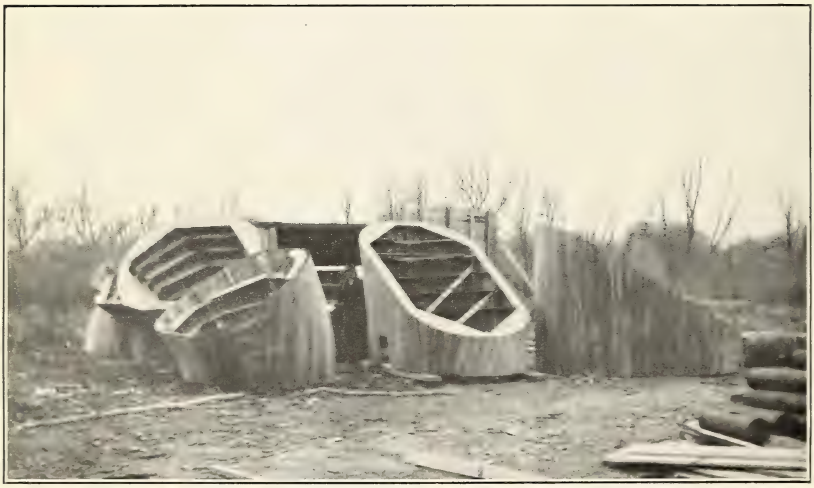


FIG. 2.—WOODEN FORMS FOR CONCRETE SUCTION PIPES FOR PUMPING PLANT ON MUSCATINE-LOUISA LEVEE AND DRAINAGE DISTRICT.

It is considered that it will be of value to introduce at this point the results of very careful and extensive investigations of rainfall and run-off on pumping drainage districts in southern Louisiana. While various local conditions are different, it is believed that for the six summer months mentioned above the relations between the rainfall and the run-off are quite similar; that is, in both sections the same percentage of the rainfall would appear as run-off. A total of 45 heavy storm periods occurring in southern Louisiana were therefore selected and the results averaged. The average precipitation during the storms selected was 4.90 inches over an average period of six days. The average run-off from these storms was 2.45 inches, distributed over a period of 7.5 days. An increase of another day in the period over which the run-off was taken would have increased the total run-off very little, while a decrease of a day in the average period of run-off would have decreased the total run-off very materially. Very few of the storms included in this list occurred after a prolonged dry period, while perhaps 20 per cent of them followed periods of rainfall that had saturated the soil. Therefore the result undoubtedly is somewhat too large, and if provision be made for such a percentage any error will be on the safe side. It will be noted that the period covered by the run-off figures is one and one-half days longer than the length of the average storm. The districts from which these records were taken average about 2,500 acres in area. As most of the districts in the upper part of the Mississippi Valley are much larger than this, it is assumed that the time required for the run-off to reach the pumping plant will be somewhat increased; therefore in the discussion following the difference between the length of the storm period and the length of the period during which 50 per cent of the rainfall would run off is assumed to be two days instead of one and one-half days, as found for the districts in southern Louisiana. It will be noted in the above table for storm intensities that a 4-inch rainfall during a three-day period occurs a little less frequently than once in two years. Decreasing this frequency by one-third, as explained above, a 4-inch rainfall in a three-day period would occur in the growing season once in three years. Fifty per cent of it, or 2 inches, would appear as run-off in five days. The average storage capacity of the canals and ditches on such districts is about 0.25 inch; thus 1.75 inches of water would have to be pumped in five days to prevent flooding, requiring a pumping capacity of at least 0.35 inch per 24 hours.

If it were considered that flooding of the lower portions of the district to a small depth for 24 hours once in three years would not be greatly injurious, the time for removing the water could be made six days, requiring a capacity of 0.29 inch. It is believed that a capacity of about 0.30 inch per 24 hours would be satisfactory on most districts. Districts of less than 5,000 acres might better increase this somewhat, while it is possible that districts having more than 15,000 acres could decrease this capacity with safety. By applying a similar process to any other storm intensity, the necessary capacity to prevent flooding could be approximated. In cases where especially valuable crops were to be raised it might well pay to provide for storms that occur only once in 10 years.

While run-off data on pumping districts in this section were not extensive enough to draw any very definite conclusions, the majority of the districts have a capacity of plant sufficient to remove a depth of 0.25 inch in 24 hours. On most of the districts this capacity has given satisfaction.

LOCATION OF PUMPING PLANT.

In general the pumping plant should be so located that the drainage water may be brought to it with the least expenditure for interior ditches. However, a more expensive ditch system will often permit the combination of several small districts into one large one, thus reducing very materially the charge for pumping-plant construction and operation. The expenditures should be adjusted so that the annual charge for interest on investment, depreciation, and operation of pumping plant will be a minimum. Where convenient, there is some apparent advantage in locating the plant on a suitable arm of the river rather than on the main river channel. The location should be so chosen as to be most easily accessible for the delivery of fuel and for convenience of attendance. Thus, in the Pekin-La Marsh District the pumping plant is situated at the upper end of the district, so as to be close to the town of Pekin.

TYPES OF PUMPING MACHINERY.

Since pumps are to be depended upon to remove promptly surplus water whose presence may endanger a whole crop, it is of the utmost importance that the pumping plant be able to work with certainty and reliability whenever needed.

The annual cost of a pumping plant is made up of three main items: (1) The charge for interest on the original cost of the plant and for depreciation; (2) the cost of fuel, or electricity if motors are used; and (3) the cost of labor. Certain general principles concerning the mutual relations of these different items must be understood before taking up the comparison of the different types of machinery available for pumping.

The interest upon the original cost of the plant goes on every year whether the plant is operated much or little. Depreciation also takes place whether the plant is operated or not, and for such installations as are under discussion it probably does not depend much upon the length of time the plants are in operation each year. What amount to allow for depreciation is a difficult matter to decide The length of time before such machinery will wear out depends chiefly upon the original quality of the installation and the care that is taken of it afterwards. We can not safely assume that such machinery will last more than 20 or 25 years. Much of it will be discarded sooner, either because it is outgrown or worn out. With proper care it is probable that such machinery may be made to endure twice as long as it will last if it is carelessly treated. It is customary to include with depreciation an item for repairs. This should be very small during the first few years of the life of a well-constructed plant, but will grow larger as the plant becomes older.

The cost of fuel per year depends to a considerable extent upon the kind of machinery used and also upon the amount of time the plant is run. Whenever a steam plant starts there is a certain consumption of fuel in preparation before starting, the heat value of which is lost when the plant is stopped. Hence, since the operation of a pumping plant is very intermittent, the fuel consumption is not strictly proportionate to the time run. The amount of fuel used also depends largely upon the skill of the fireman. An unskillful man may easily use 25 per cent more fuel than is really necessary. Different types of equipment require different amounts of fuel to produce the same result; that is, some types are more economical of fuel than others. The cost of the labor at the pumping plant depends upon the class employed and the length of time it is required. The cost of electricity per year depends on the type of pump and the care used in installation, as well as upon the total time the plant is run. The distance from the transmission lines of the large power companies will influence the power rate. On the larger districts the power rate is usually less than on the smaller ones.

In general, in a given location, that type of pumping plant will be best which requires the least annual cost, including all the elements enumerated above. To a certain extent, variations in these different elements tend to offset each other. For example, a highly efficient plant from the point of view of fuel economy is complicated and expensive, and hence will have a relatively high charge for interest and depreciation. Likewise, it will require skillful and high-priced attendance, but may, on the other hand, in a large plant, require fewer men. If cheap labor is employed, the coal and repair bills will be increased and the deterioration of the plant will be more rapid. The advantage of fuel economy is relatively more important in locations where fuel is especially high, and in a large plant than in a small one, but in any plant it diminishes in importance as the time of running per year decreases or becomes much interrupted.

The above principles have long been regarded as fundamental in the science of mechanical engineering design. Applying them to drainage pumping, where the work is usually confined to a small portion of the year, the labor available is cheap and unskilled, and the other conditions are not favorable for effective care of the plant, the fundamental requirements in a drainage pumping plant are reliability and simplicity.

The centrifugal pump is the kind most often used for drainage. It is simple, has no valves, and can be obtained in all sizes. Its disadvantages are, that it must be primed in starting, that it can operate satisfactorily under a given set of conditions only within a narrow range of speed, and that its efficiency when operated under practical conditions that obtain in drainage pumping is not high. Recently pump manufacturers have given more attention to the design of centrifugal pumps for drainage pumping, and the newer pumps are better fitted for such service. In order to work economically and to maintain a uniform capacity, the speed must be varied with the changes in the total head under which it operates. The size of the centrifugal pump is usually given as the diameter of the discharge opening—for example, a 24-inch pump is one having a discharge pipe 24 inches in diameter as it leaves the pump. Most large pumps are made with a horizontal shaft and are called horizontal pumps. When larger than 24 inches they frequently have a double suction, thus bringing water to both sides of the impeller; this has the advantage of balancing the side thrust on the impeller and shaft. Without this arrangement some automatic balancing device must be used in connection with the impeller and a collar thrust bearing on the shaft.

The capacity of a pump is usually reckoned by assuming the water to have a velocity of 10 feet per second through the discharge opening of the pump. However, some manufacturers base the rated capacity of their pumps upon an assumed velocity of 12 feet per second. While it is possible to obtain a velocity of 10 feet per second through the discharge opening of the pump by the use of proper speed at the various heads, as the pumps are usually operated in practice the discharge is often 20 per cent less than the theoretical rating. This is due to underestimation of the power required for the various heads pumped against; as the head increases the speed must also be increased. During the high-water stages of 1913 on the Illinois River several plants had their capacity reduced about 50 per cent because the engines were not large enough to turn the pumps at the requisite speed. If enough power is furnished, the capacity of the centrifugal pump can be pushed as much as 25 per cent above its rated capacity with a relatively small loss in efficiency. Tests of such plants have been made which showed the amount pumped to exceed the rated capacity by 50 per cent, but at this overload the efficiency of the pump is but a small part of its best figure.

Rotary displacement pumps and plunger pumps have occasionally been used for pumping drainage water, but they are not adapted to this use. They may have a very high mechanical efficiency, but their cost more than offsets this advantage. Scoop wheels have been extensively used in England and Holland, and to some extent in this country in Louisiana. They may be made to have a higher efficiency than centrifugal pumps, especially for very low lifts, but they are not adapted to locations where the head is subject to wide fluctuations. A full description of the construction and operation of these scoop wheels has been given by W. W. Wheeler.[1] Another publication [2] describes a recent large and sucessful installation in Holland, and gives the results of tests.

Recently large screw pumps have been installed for the pumping of drainage water from districts in Louisiana. One district of about 9,000 acres has installed and is operating two such pumps, each having a diameter of discharge pipe of 78 inches. The city of New Orleans recently contracted for and is now installing 12 such pumps for the disposal of drainage water; these have diameters of discharge of 12 feet. The impeller in the screw pump is a segment of a screw and is somewhat similar to a boat propeller, except that it has many more blades. On the discharge side diffusion vanes take the water from the blades of the impeller and change its partly spiral motion to one which is parallel with the axis of the discharge pipe. It is expected that such a pump will operate more satisfactorily under changing heads than will a centrifugal pump, and for very large installations it is cheaper than a centrifugal pump of equal capacity.

SOURCES OF POWER.

Steam, gas, and electricity are all used as sources of power for pumping plants. In Illinois soft coal is abundant and cheap, though in some places crude oil or natural gas may be found more available. A suction gas producer with a gas engine has been tried, but such a plant requires hard coal or coke for fuel, which greatly increases the expense. Furthermore, such a type of plant is not adapted to intermittent working. Electricity is by far the most convenient source of power where it is available, but in general it will be considerably more expensive than using coal to produce steam. Its convenience will perhaps offset its cost, especially for a small plant. In the latter part of this bulletin, under the heading "Amount and cost of pumping," data from a number of districts are given which show the comparative cost of power in the various districts. These data indicate that where cheap coal or gas is available steam is the most economical source of power.

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[1] The Drainage of Fens and Low Lands by Gravitation and Steam Power. London and New York, 1888.

[2] Engineering News, 63 (1910), No. 20, p. 581.

Return tubular, marine, locomotive-type, and water-tube boilers have all been used. For a small plant the locomotive type or the simple return tubular boiler is simplest and cheapest. For plants of over 200 horsepower water-tube boilers have decided advantages, though they may cost somewhat more than the simpler fire-tube boilers. The boilers of all large plants should be kept constantly insured by some reliable boiler insurance company.

KIND OF ENGINE OR MOTOR.

Either vertical or horizontal engines may be used. When possible, it is best to have the pump directly connected to the engine. This is usually not feasible for a pump smaller than the 24-inch size, because such a pump would require too high a speed of the engine. In this case the pump must be driven by a belt or a rope. The engine should be condensing and, if large, should be compound. A high-speed automatic engine is more difficult to operate with inexperienced labor than a slow-speed engine, but it has the advantage of being smaller and lighter for the same power than the slow-speed engine of the Corliss or some similar type. In general, however, the high-speed engine is not as economical of steam as the slow-speed type. The engine should be provided with a governor able to keep it from racing if the entire load is suddenly removed. The engine should be adjustable for a considerable range of speed, should be able to carry a considerable overload, and should work as economically as possible through a wide range of load. As will be explained more fully below, it is especially desirable that an engine for this work should be capable of very flexible operation.

If a motor is used to drive the pump, it should be of the constant-speed slip-ring type. If the motor is direct-connected to the pump or is connected by some means that does not allow the speed ratio to be changed, the pump will work satisfactorily at only one head; that is, the head suitable to the fixed speed. If the lift be higher, the quantity pumped will be much reduced, while if the lift be lower the quantity will be much increased, the efficiency lowered, and danger of overloading the motor will be incurred. Manufacturers have recently placed on the market a centrifugal pump with a modified screw-type impeller which requires about the same horsepower for a rather wide range of heads. The screw pump previously described also has this feature. Both the screw pumps and the centrifugal pumps with the modified screw-type impeller have been connected to motors with a fixed-speed ratio, such as by the use of gearing, and have worked satisfactorily. However, as the head increases the quantity pumped must decrease. Along the upper Mississippi and the Illinois River the greatest demand on the pumping plant usually occurs at the time when the head pumped against is greatest. If the speed of the pump be fixed to give full capacity at the greatest head, at lower heads the discharge of the pump will be greatly increased and the efficiency much lowered. As the average lift will be much lower than the maximum, usually scarcely half, the yearly efficiency of the pump will be small and the consumption of power large. By far the better method is to connect the pump and the motor by means of a belt with three sizes of pulleys or with silent chain drive with two or three sizes of sprocket wheels. The pump speed can then be altered to suit the head. In Louisiana, where the fixed-speed ratio is used, the greatest demand on the pumping plant occurs at the lowest head, and as the head increases the amount of water to be pumped greatly decreases; consequently the capacity of the pump fits the need.

AUXILIARIES.

The steam plant will require the usual auxiliaries. A surface condenser will furnish pure feed water, but some form of jet condenser is simpler and cheaper. For priming the pumps a steam ejector is the simplest arrangement, though an air pump may be used. Separators and traps, filters and heaters, should be used as needed for safe and economical operation. Piping and boilers should be well covered, to reduce as much as possible the amount of radiation and condensation of steam.

In large plants boilers, engines, and pumps should all be divided into two or more equal-sized units, so arranged that they can, when necessary, be operated independently, and the auxiliaries should be installed in duplicate.

CALCULATION OF THE SIZE OF PUMPING PLANT.

To illustrate the method of determining the size of machinery to be installed in a pumping plant, the calculation will be made for a district of 10,000 acres. To remove a depth of water of one-fourth inch over the whole district in 24 hours will require a capacity of 105 cubic feet per second. For such a district it will be best to use two separate pumps, which should be identical in size. A 30-inch pump will be rated at about 49 cubic feet per second, a 32-inch pump at about 56 cubic feet per second, and a 36-inch pump at about 71 cubic feet per second. In this case it would be best to use two 32-inch pumps, which would seem to have a slight margin of excess capacity. However, since pumps of this style so frequently fail to have the capacity assumed, they should be taken only under the guaranty of the manufacturer and subjected to a thorough test after they are installed. The pumps will need to be run at about

200 revolutions per minute, and hence may be directly connected to two separate engines.

Assume that the difference between the river-surface level outside the district and the desired ground-water level within the district will vary from a possible maximum of 20 feet during spring floods to nothing in the late summer. The machinery must have sufficient power to pump against the highest head, although much of the time it will run against a lower head. This is a disadvantage, so far as the efficiency of the machinery is concerned, because every engine runs most efficiently at about its rated capacity, and if it is either overloaded or underloaded its efficiency is reduced. The actual head against which the pumps must force the water, sometimes called the dynamic head or the hydraulic head, is always greater than the difference in water level as described above, which is called the static head. This increase is due to the friction of the water in the suction and discharge pipes and the so-called velocity head—that is, the head necessary to start the water into motion in the pipes. The friction in pipes carrying water at a high velocity is greatly affected by the kind of pipe used and the smoothness of the interior surface, and hence can not be predicted with certainty. For a velocity of 8 feet a second the friction in 200 feet of 48-inch pipe would probably amount to 2 feet of head and in a 24-inch pipe to at least 6 feet of head. A bend of 90° adds probably a foot to the friction head. The velocity head amounts to about 1 foot, and if the water passes around a sharp edge in entering the suction pipe there is probably a loss of another foot of head. To be safe, then, we should assume the addition of at least 6 feet to the static head to give the head on the pump. Another small amount might be added to the lift on account of the lowering of the water level in the supply ditch when the pumps are being operated. But it is probably better to ignore this last addition and assume that during the short time when the river is at maximum flood stage the water level in the district will not be pumped down quite to its normal level rather than to provide extra capacity for the very short and infrequent periods of maximum river floods.

With a total head of 26 feet the hydraulic horsepower is calculated by multiplying together the lift (26 feet), the number of cubic feet of water per second (105), and the weight of 1 cubic foot of water (62.5 pounds), and dividing by 550. In this case the result is 310. The power required of the engine will be greater than this amount, depending upon the efficiency of the pumps, which for these conditions should be at least 60 or 70 per cent. For this case probably manufacturers would be willing to guarantee an efficiency under test of 70 per cent. Accordingly the maximum brake horsepower required of the engines would be 440. Engines suitable for this work should be able to carry continuously an overload of 25 per cent without an increase of speed and without a great loss in steam economy; hence, if 440 horsepower puts this overload on the engines, their normal rating would be 350 brake horsepower. They should develop sufficient power at the speed at which the pump has the best efficiency for a static lift of 10 feet or a dynamic lift of 16 feet, in this case about 270 horsepower. When the pumps operate at the maximum lift, their speed should be increased in order to maintain the discharge and the efficiency of the pumps, and the engines should be adjustable and so constructed as to run safely at the higher speed. Since increasing the speed of itself increases the power developed by the engines, this gives a margin of safety in the capacity of the engines. If two separate engines and pumps are used, each engine should have a brake horsepower of 175. Its indicated horsepower will be from 10 to 20 per cent greater. An engine should run fairly economically down to three-fourths of its rating. These units, then, would be economical of fuel down to a static lift of 10 feet. Records of plant operation on districts on the Illinois River show that this is quite near the average pumping head. As the above plant is designed, when the lift falls below 5 or 6 feet the steam used by the engine will be much increased in proportion to the useful work actually performed in lifting the water. This disadvantage is not very important for plants on the Illinois River, but for districts on the Mississippi River, at least for those above Burlington, Iowa, where records were available, it is of very great importance, as the mean lift is quite often less than 5 feet. Records from the Louisa-Des Moines Drainage District No. 4 show the following average static lifts: 1910, 4.20 feet; 1911, 2.82 feet; 1912, 5.20 feet; 1913, 5.74 feet. As will be shown later in this bulletin, under the heading ''Amount and cost of pumping,'' the cost per unit of water lifted 1 foot was greatly increased in 1911, when the lift was very low. Where the average lift is as low as 5 feet and the maximum static lift is as great as 20 feet, it would be very difficult to install an engine that would work with economy at the low head and deliver the full capacity of the pump at the highest head. In the above example, for service on the Mississippi River it would be better to install a smaller engine than one with a nominal rating of 350 brake horsepower and to work it at a heavier overload than 25 per cent at the maximum head. Such an engine would then work at about three-fourths of its nominal rating at a static lift of 6 feet and would not lose much in economy of steam consumption.

In case it were desired to drive the pump by a motor, the calculations for the maximum required horsepower would not differ from those in the foregoing example. However, as the motor is not capable of an overload its nominal rated horsepower must be large enough to provide for the maximum demand. The question of adjustment of speed has already been discussed.

If it were desired to provide for a greater depth of water removed per 24 hours, the proper additions could be made to the above results.

The boiler capacity required would depend upon the particular engines used, upon the number and kind of auxiliaries, and upon the quality of coal to be used. For a plant of this kind, however, the nominal boiler horsepower should be at least as large as the rated engine horsepower. The boilers must have a capacity amply sufficient to supply safely all the steam needed by the engines when operating at their maximum overload. This may be secured by additional boilers or by forcing the boilers, when necessary, through appliances for producing an artificial draft. In general, large engines of the best types require less steam per horsepower than smaller ones. In a very large plant the nominal boiler horsepower may be less than the engine horsepower.

In designing such a plant, all the elements must be carefully calculated to secure in each the necessary maximum capacity. It is especially needful that the auxiliaries be large enough. A deficiency in some single comparatively slight but essential element may hinder the operation of the whole plant. Durability and reliability in workmanship and materials may be secured to some extent by specifying that the boiler shall be capable of operating at a higher pressure than its regular working pressure, and that the engines shall be able to run with higher steam pressures and at higher speeds than the normal. Descriptions of a large number of pumping plants and the results of tests are given in detail in a previous bulletin.[1]

BUILDINGS AND FOUNDATIONS.

The machinery should be inclosed in a substantial and durable building. The foundation of the building, and especially of the machinery, should be prepared with great care. As the soil conditions are very likely to be bad, careful investigations should be made by means of borings. Nothing should be assumed. Failure to make such borings has many times resulted in great difficulties during construction, and in at least one case in failure after completion. The results of such investigations will sometimes show that the proposed site of the plant must be changed. Drainage district officials who are responsible for the construction of the plant should not only make certain that the proposed foundation will prove satisfactory under the prevailing soil conditions, but should allow only those to do the work who are experienced in building foundations under such adverse conditions. Inexperienced contractors often have underbid others who have become familiar with foundation construction under such circumstances, and the former have been awarded the contract. However, such bids often prove to be the highest, because of delays, unsatisfactory results, and even forfeiture of contract. Often the expense of foundation is a large part of the cost of the completed plant. When properly constructed it is the most permanent part of the plant, and should outlast the building and serve as the support for several sets of machinery.

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[1] U. S. Dept. Agr., Office Expt. Stas. Bul. 183, Mechanical Tests of Pumps and Pumping Plants used for Irrigation in Louisiana in 1905 and 1906.

ARRANGEMENT OF SUCTION AND DISCHARGE PIPING.

Proper arrangement of the suction and discharge pipes is essential to the economical and reliable operation of the plant. The pipes should be as direct as possible. All bends should be of long radius, and the interior of the pipes should be as smooth as practicable, in order that the lost work, due to friction, may be small. Since the head consumed in overcoming friction varies as the square of the velocity, the larger the piping the less the friction loss. The piping should be made large enough to reduce the velocity in the pipe to as near 5 feet per second as practicable. All changes in size and direction of the piping should be gradual, and at no point should the water be permitted to flow around a sharp corner.

The suction pit should be carried to a greater depth than the main canal, even though this results in some danger of excessive seepage and of opening springs or boils through the lower layers from the river when the suction pit is close to the river bank. This has been a frequent source of difficulty in pumping plants, and on this account the discharge pipe should be made longer than it often is. The sides of the suction pit should be protected by a reinforced-concrete wall supported by round piling, and usually a cut-off wall of sheet piling should be driven along the center lines of these side walls. If the material in the bottom of the pit is inclined toward quicksand, or if there are springs and boils, the bottom as well as the sides of the pit should be of reinforced concrete well tied down by the use of round and sheet piling, as it must be remembered that there will be an upward pressure on the bottom of the pit when the water is lowered. The pit should be shaped to form a suitable gradually-curved entrance to the suction pipe, and should be provided at its outer end with a steel screen. The screen openings should be of such size that the velocity through them, under normal conditions, would not be more than 1 foot per second.

In order that the suction lift may be kept low, the pump should be set as low as is consistent with securing good foundation, ease of installation, and safety after installation. In general, the plant should be at such an elevation as would make it very unlikely that excessive rainfall or suspension of pumping would cause the water ever to rise high enough to interfere with its operation. In a certain district on the Illinois River it became necessary to protect with a levee the pumping plant and its immediate grounds from the remainder of the district.

There are various arrangements of pipes for taking the water from the suction pit to the pumps, but there are a number of features in addition to those already mentioned which apply to most of the designs. The suction pipe should be perfectly air-tight and so strong as not to collapse under the greatest suction that will be put upon it. The entrance should be expanded so as to provide for a gradual contraction of the stream of entering water. The curvature of the expanded end may be relatively abrupt, but should be smooth, so that the water shall not flow swiftly past any sharp edge or angle. In general, in contracting a stream there is no loss of head, even if the reduction in cross section be comparatively sudden, so long as the surfaces are smooth and without sharp angles. If the entrance end of the pipe is cut horizontally, the edge must be at least as low as the lowest level to which the water in the suction pit is to be reduced, and it would be better if it were a foot lower. Below this edge there should be a clear distance to the bottom of the pit equal to one or one and one-half times the diameter of the suction pipe. If the entrance of the suction pipe is expanded to such a degree that the velocity of the incoming water is not more than 2 feet per second, the water level can be drawn down to the end of the suction pipe without trouble from sucking air. Without such expansion, while the level of the water is still considerably above the end of the pipe, much air will be drawn into the pump, with a consequent loss of efficiency and danger of losing the priming. Neglect to provide ample waterway around the entrance to the suction pipe has been a source of much annoyance to pumping plants. If the end of the suction pipe is cut vertically the lower side of the pipe should be at or near the bottom of the pit. As the water can not be lowered to a level very near the top of the pipe without the latter sucking air, the entrance should be oblong in shape, with its greatest diameter horizontal. Small whirlpools will often form over such pipes and allow air to enter. This condition is likely to give the most trouble as the water nears the top of the pipe but before there is any appreciable velocity of the water in the suction pit toward the pipe. When the water is low enough in the pit to have an appreciable velocity toward the pipe the small whirlpools will be swept into the pipe before they are of sufficient size to admit air. Where these whirlpools cause the pumps to lose their priming a remedy has been to place a large raft of timbers in the pit in such a manner that whirls can not form directly over the end of the pipe.

One arrangement of pipe which is much used is to place the suction pipe on a uniform slope from the pump to its lower end in the suction pit. This slope should be flat enough to allow of the pipe being supported rigidly. The end of the pipe can be cut horizontally, making an oblong opening, or the end of the pipe can be turned down in an elbow of large radius; such an arrangement is used on the plant of the Louisa-Des Moines Drainage District No. 4. (See fig. 2.) Again, the pipe can be curved until it is horizontal and the end cut vertically. Another arrangement is to bring the suction pit directly under the pumps and have a short vertical pipe with an expanded horizontal end. Especially if there are two or more pumps close together, the area of the suction basin will be rather limited and a great many cross currents will be set up. When such pumps are in operation this turbulent condition of the water in the suction pit is very noticeable. The efficiency of the centrifugal pump is seriously affected if the water approaches the impeller in this condition. Riveted steel pipes are universally used for the above-mentioned designs. The metal in the pipes should be special low-carbon steel, so that it will better resist corrosion. After erection the pipes should be heavily coated with an asphalt paint.

The use of reinforced concrete for suction pipes is just commencing. Figure 3 shows the plan of the pumping plant being constructed on the Muscatine-Louisa Levee and Drainage District, which has concrete suction pipes built into the foundation. As the suction elbows on the pumps, and the yoke which connects them with the concrete pipe, are of cast iron, this construction of suction pipe should be permanent, with very little if any depreciation. Figure 2 of Plate VIII shows the forms for these concrete pipes. It will be noted from figure 3 that these pipes do not extend into the suction pit; consequently the water, as it approaches the entrance to the pipes, has no chance to pass the edge on the outside of the pipe and cause eddies. As the edges of the entrance are rounded the water that approaches along the wall or bottom of the suction pit does not have to turn a sharp angle. The entrance losses for these pipes should be less than in any of the others described and the water should approach the impeller of the pump in a condition favorable to the best pump efficiency.

The discharge pipe also should be strong enough to resist any suction pressure that may be put upon it. It should be gradually expanded, immediately after leaving the pump, so as to reduce the velocity to as near 5 feet per second as seems practicable. It must be remembered, however, that in expanding the cross section of a stream so as to reduce its velocity the changes must be very gradual; otherwise there will be loss of energy due to the impact of the rapidly moving stream against the more slowly moving mass of water.

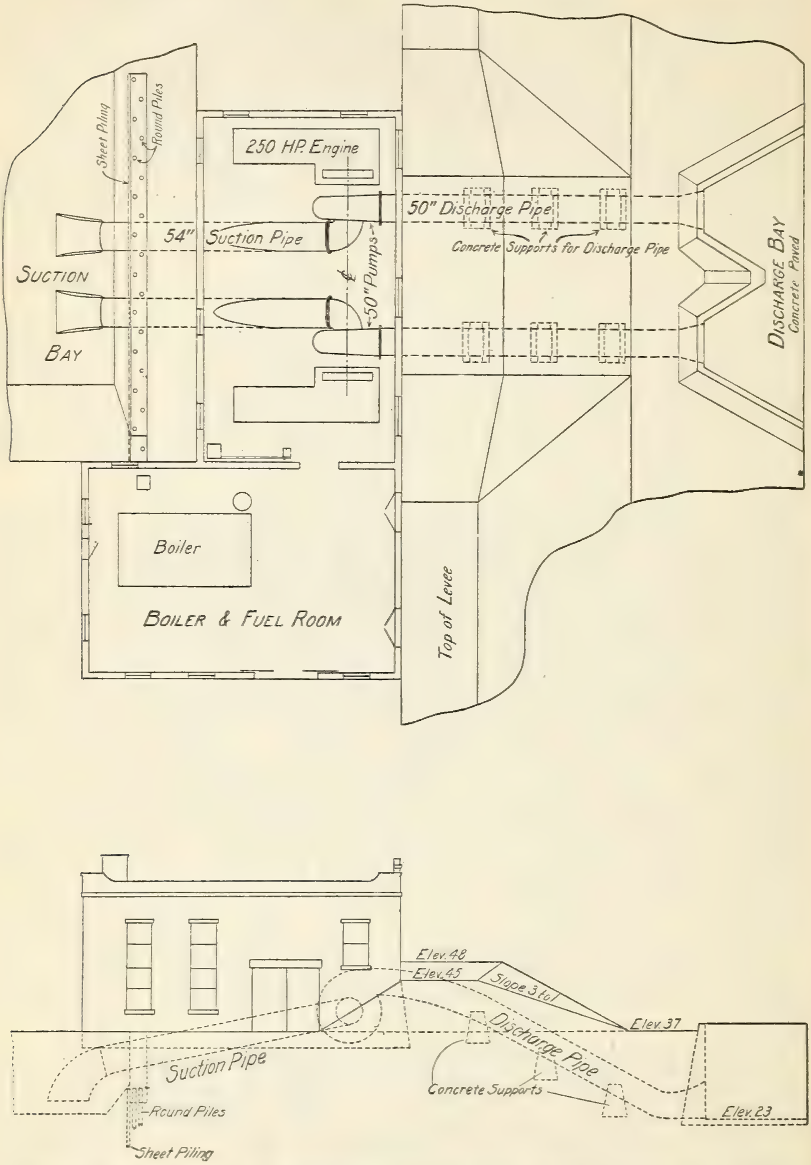

FIG 2.—Plan and elevation of pumping plant, Louisa-Des Moines Drainage District No. 4, Iowa.

Practice is not uniform as to the best method of preventing water from flowing back through the pipes into the district from the river when the pumps are not operating. Some prefer to carry the discharge pipe through the levee above high water and then down, with its end opening below low-water line, so as to secure a water seal. This is necessary in order that the pipe may act as a siphon and that hence the static head on the pump may be only the difference in level between the water in the canal and the water in the river. In this case the pipe must have an opening at the highest point for use in priming the pump. This opening will also be used to admit air to break the seal, thus preventing water from flowing back by siphon action when the pump is stopped. If the pumps equipped with this type of discharge pipe are driven by motors, a flap valve should be used on the end of the pipe to prevent the water from siphoning back through the pump while it is being primed, thus turning the motor backward and overloading it. This valve need not fit tightly and

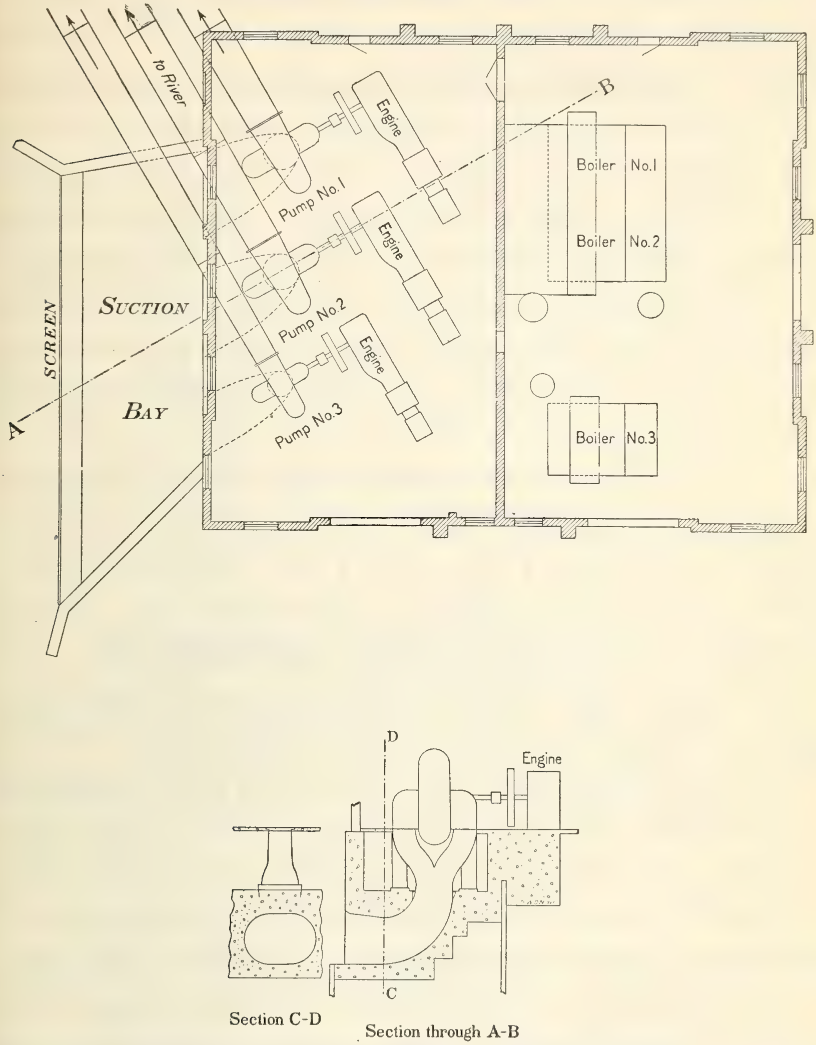

FIG. 3.—Plan and elevation of pumping plant in Muscatine-Louisa Levee and Drainage District, Iowa.

will be of use principally when the water in the river is high. Some arrangement should also be made for continually exhausting the air from the highest point in the discharge pipe; otherwise air will collect, the pipe will not run full, and the perfect siphon action will be interfered with. By connecting the top of the pipe with a large receiver under vacuum, equipped with a water-gauge glass, the condition of the pipe can always be easily observed and pumping of air will be only necessary at intervals.

Others prefer to carry the discharge pipe through the levee on a level. This necessitates the installation of a gate valve in the pipe next to the pump. If the pipe is not over 2 feet in diameter, a flap valve might be used on its outer end; but such a valve is never entirely reliable and certain in its operation, and its use is not recommended. A gate valve must be of the slow-closing type, as it is dangerous to the plant to close such a valve quickly. If the pipe is carried through the levee, all the precautions that were mentioned in the section on gravity sluiceways should be used to prevent seepage along the pipe. The outlet end of the pipe must have adequate protection against river wash and against undercutting by the water discharged by the pump.

Attention has already been called to the plans of the Louisa-Des Moines and the Muscatine-Louisa pumping plants (figs. 2 and 3). Plate VI and figure 1 of Plate VIII are views of the Louisa-Des Moines pumping plant. Figure 1 of Plate VI is a view from outside the levee, showing the front of the building and the discharge bay. Figure 2 of the same plate is a view of the engine room, giving a good idea of the size of the 50-inch centrifugal pumps, each of which is direct-connected with a tandem compound condensing engine of 250 horsepower. Figure 1 of Plate VIII is a view of the rear of the building, showing the levee beyond and the enlarged ends of the suction pipes.

Plate III, figure 2, and Plate VII are two views of the Lazwell plant of the Des Moines County Levee and Drainage District No. 1. Plate III, figure 2, is a view of the suction side of the plant, showing concrete suction basin, steel screen across the portion of the pit where the water goes into the pumps, and the discharge pipes curving out of the building, over the levee, and downward. Plate VII is a closer view of the discharge pipes as they turn down through the top of the levee to the river channel to the left. These views illustrate the easy curves used in such pipes. In Plate VII the small pipes connected to the top of the discharge pipes are used in priming the pumps and also in exhausting the air that collects in the top of the pipes while the pumps are operating.

AMOUNT AND COST OF PUMPING.

Table 12 shows the yearly rainfall and amounts pumped for the four districts already mentioned under "Necessary capacity of pumping machinery." It will be noted that there is considerable variation in the relation between rainfall and amount pumped for different years on the same district, and for the different districts during the same year. The average percentage of the rainfall pumped was 40.6 per cent if based upon the total yearly rainfall, or 46.7 per cent considering only the rainfall during the months when pumping was going on. The average annual rainfall for the above four stations for all the years available was only 26.21 inches, while the average annual rainfall for the Weather Bureau stations at Muscatine, Peoria, and St. Louis is 37.39 inches. Assuming that the ratio would be the same, 40.6 per cent, if the rainfall were normal, we might expect the amount to be pumped as a depth of 15.18 inches. Moreover, as the percentage of the rainfall appearing as run-off is usually greater during wet years than in dry ones, the above figure would more likely be less than the average run-off rather than greater. Those districts which do not have gravity sluices may well expect to do more pumping than those which have, although most of the rainfall and pumping occur at the time of the year when the rivers are high.

TABLE 12.—*Annual rainfall and amounts pumped.*

District.	1912			1913			1914			Total.			Per cent of rainfall in run-off.	
	Rain-fall.[1]	Rain-fall.[2]	Run-off.	Rain-fall.[1]	Rain-fall.[2]	Run-off.	Rain-fall.[1]	Rain-fall.[2]	Run-off.	Rain-fall.[1]	Rain-fall.[2]	Run-off.	Rain-fall.[1]	Rain-fall.[2]
	In.	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*	*In.*		
Louisa-Des Moines	25.67	15.07	15.15	28.16	14.84	11.32	......	......	......	53.83	29.91	26.47	49.1	88.5
Pekin-La Marsh	34.49	34.49	13.41	26.05	26.05	11.80	20.85	20.85	4.87	81.39	81.39	30.08	37.0	37.0
East Peoria	......	......	......	......	......	......	22.75	22.75	6.98	22.75	22.75	6.98	30.8	30.8
Coal Creek	......	......	......	......	......	......	25.50	25.50	10.98	25.50	25.50	10.98	43.6	43.6
Average	30.08	24.78	14.28	27.11	20.44	11.56	23.03	23.03	7.61	26.21	22.79	10.64	40.6	46.7
Per cent of rainfall in run-off	47.5	57.6	......	42.6	56.6	......	33.0	33.0	......	40.6	46.7	......	......	......

[1] Total rainfall for year. [2] Rainfall for months during which drainage was secured by means of pumps.

The cost of operation of pumping plants for drainage pumping will vary largely according to type of machinery and the method of operation. While the cost of operation of various types of machinery is well understood, the conditions obtaining on the average drainage district are likely to affect seriously the accepted cost figures. In view of this fact cost data were obtained from a number of drainage pumping plants now in operation. The operating expenses were classified under "Labor," "Fuel," "Repairs," "Supplies," and "Superintendence." Tables 13 and 14 give the cost of such items, per acre per year, for a number of districts. As some districts have a greater proportion of hill water than others, the cost has been shown for the assessable area and for the total drainage area of each district. The tables state the number of years of operation the cost figures cover, and if not otherwise stated, 1914 is the last year included. Table 13 is for steam-driven plants and Table 14 for electrically-driven plants.

It will be noted that the average yearly cost per acre of total drainage area for the steam-driven plants is $0.43, and that the variation in cost is from a maximum of $0.93 for the Coal Creek District to a minimum of $0.15 for the Des Moines County No. 1 District. Of course, great variations in the amount of water pumped and in other local conditions tend to affect the relative costs for the various plants. Where the pumps are driven by motors the average yearly cost per acre of drainage area is $0.88, with a maximum of $1.07 for the Pekin-La Marsh District and a minimum of $0.63 for the Big Swan District.

TABLE 13.—*Costs of operation—Steam-driven plants.*

District.	Number of years' operation.	Average cost per acre per year.											
		Labor.		Fuel.		Repairs.		Supplies.		Superintendence.		Total.	
		A.[1]	T.[2]	A.	T.	A.	T.	A.	T.	A.	T.	A.	T.
Nutwood	5	$0.115	$0.082	$0.158	$0.113	$0.015	$0.011	$0.038	$0.027	$0.079	$0.057	$0.405	$0.290
Eldred	1	.129	.126	.153	.150	.019	.017	.028	.027	......	......	.329	.320
Hillview	5 (1913)	.190	.126	.278	.185	.047	.031	.032	.021	.114	.076	.661	.439
Big Swan	7 (1913)	.123	.098	.250	.200	.032	.026	.031	.025	.071	.056	.507	.405
Meredosia	9	.100	.069	.314	.238	.023	.017	.020	.015	.073	.055	.530	.394
Crane Creek	4	.173	.154	.281	.250	.014	.012	.049	.043	......	......	.517	.459
Coal Creek	5 (1913)	.331	.300	.598	.540	.049	.044	.041	.037	[3] .012	[3] .010	1.031	.931
Lacey	4 (1905)	.072	.072	.247	.247	.030	.030	.017	.017	......	......	.366	.366
Do	2 (1910)	.151	.151	.416	.416	.017	.017	.027	.027	......	......	.611	.611
Spring Lake	5	.181	.103	.306	.175	.024	.014	.081	.046	.138	.079	.730	.417
Louisa-Des Moines	2 (1913)	.164	.133	.204	.165	.041	.033	.027	.022	.048	.039	.484	.392
Des Moines County No. 1	2 (1913)	.140	.053	.162	.062	.034	.013	.033	.012	036	.014	.405	.154
Average	....	......	......	......	......	......	......	......	......	......	......	......	.432

[1] Based on assessable area.
[2] Based on total drainage area.
[3] Partial.

TABLE 14.—*Costs of operation—Electrically-driven plants.*

District.	Number of years' operation.	Average cost per acre per year.											
		Labor.		Power.		Repairs.		Supplies.		Superintendence.		Total.	
		A.[1]	T.[2]	A.	T.	A.	T.	A.	T.	A.	T.	A.	T.
Big Swan	1	$0.116	$0.098	$0.592	$0.473	$0.000	$0.000	$0.000	$0.000	$0.071	$0.056	$0.779	$0.627
Lacey	2	......	......	[3] .740	[3] .740	......	......	......	......	......	......	.740	.740
Perkin-La Marsh	3	.132	.107	[3] 1.160	[3] .940	.021	.018	.002	.002	......	......	1.315	1.067
Coal Creek	1	.159	.143	.860	.776	.000	.000	.010	.010	.020	.018	1.049	.947
East Peoria	1	.504	.244	1.200	.582	.001	.001	.023	.011	.372	.180	2.100	1.018
Average	....	......	......	......	......	......	......	......	......	......	......	......	.880

[1] Based on assessable area.
[2] Based on total drainage area.
[3] Cost of power calculated on basis of schedule in effect at the Coal Creek plant.

Tables 13 and 14 do not take into account the fixed charges, which include interest, depreciation, taxes, and insurance. In computing the fixed charges interest has been assumed at 6 per cent, taxes and insurance at 1 per cent, and depreciation at 6 per cent. While the rate of depreciation on the machinery might be somewhat greater than this, the cost of the machinery is only a part of the total cost of the plant, and a rate of 6 per cent is higher than would apply to the building, and especially to the foundation, the cost of which last item is often a large part of the cost of the completed plant. Tables 15 and 16 give the total cost per acre per year for the districts given. As before, two tables are made, one for steam and one for electrically driven pumps.

It will be noted that the average yearly cost per acre of total drainage area is, for steam-driven plants, $0.78, and that the variation in cost is from a minimum of $0.40 for the Des Moines County District No. 1 plant to a maximum of $1.22 for the Lacey plant. Where the pumps are driven by motors the average yearly cost per acre of drainage area is $1.38, with a maximum of $1.89 for the East Peoria District and a minimum of $0.89 for the Big Swan District. As stated above, the amount of water pumped and other local conditions tend to alter the true relation between the costs of operation of the various plants. However, from three districts accurate and detailed records were obtained, so that the cost in operating expense of lifting an acre-foot of water 1 foot could be accurately determined. Table 17 is a statement of amount pumped and the unit cost on these three districts.

Both plants on the Illinois River have a much higher mean lift than has the one on the Mississippi River. A large variation in costs is noted for the different years for the Louisa-Des Moines plant. This can be traced directly to the average lift; the pumps do not work so efficiently at the lower lift. In this respect the plants on the Illinois River have a considerable advantage over the Louisa-Des Moines plant, as their lifts are higher and the pump efficiencies must be much better. In the early part of the discussion in this section it was assumed that under normal conditions the average amount of water to be removed per year was a depth of 15.2 inches. With the unit costs as taken from the actual operating expenses of the three plants given in Table 17, the costs of drainage per acre per year have been calculated and are shown in Table 18. In selecting the unit cost to use for the Louisa-Des Moines plant, the one corresponding to the highest head is taken, so that a better comparison can be made with the plants on the Illinois River. However, the lifts are much higher for the latter plants, thus putting them at a considerable advantage; for if their lifts were as low as that of the Louisa-Des Moines plant their efficiencies would be much decreased and the unit costs correspondingly increased. The results are calculated for average lifts of 5 and 10 feet. All three of these plants are well built and carefully operated. They are quite representative of their types, and the costs of pumping given in Table 17 should quite closely approximate what might be expected in similar plants operated under like conditions.

TABLE 15.—*Total costs of pumping—Steam-driven plants.*

District.	Horsepower of engines.	First cost.	Annual fixed charges.	Annual operating expenses.	Assessed area.	Watershed area.	Charges per acre per year on assessed area.		Charges per acre per year on watershed area.		Total charges per acre per year on—	
							Fixed.	Operating.	Fixed.	Operating.	Assessed area.	Watershed area.
					Acres.	*Acres.*						
Nutwood	500	$36,480	$4,750	$4,338	10,700	15,000	$0.444	$0.405	$0.317	$0.290	$0.849	$0.607
Eldred	300	28,000	3,640	3,132	9,305	9,500	.391	.337	.383	.330	.728	.713
Hillview	300	25,000	3,250	8,140	12,300	18,500	.264	.662	.176	.440	.926	.616
Big Swan	300	25,000	3,250	6,084	12,000	15,000	.271	.507	.217	.406	.778	.623
Meredosia	200	18,000	2,470	1,995	3,760	5,000	.657	.531	.494	.399	1.188	.893
Crane Creek	250	16,000	2,080	2,759	5,340	6,000	.390	.516	.347	.460	.906	.807
Coal Creek	200	16,000	2,080	6,912	6,700	7,426	.310	1.032	.280	.930	1.342	1.210
Lacey	220	15,000	1,950	1,898	5,200	5,200	.375	.365	.375	.365	.740	.740
Do	220	15,000	1,950	1,950	3,200	3,200	.610	.610	.610	.610	1.220	1.220
Louisa-Des Moines	500	50,000	6,500	6,280	13,000	16,000	.500	.483	.406	.393	.983	.799
Spring Lake	600	60,000	7,800	8,600	12,850	22,500	.607	.670	.347	.382	1.277	.729
Des Moines County No. 1	1,230	144,000	18,700	11,570	28,600	75,000	.654	.405	.249	.155	1.059	.404
Average												.780

TABLE 16.—*Total costs of pumping—Electrically-driven plants.*

District.	Horsepower motors.	First cost.	Annual fixed charges.	Annual operating expenses.	Assessed area.	Watershed area.	Charges per acre per year on assessed area.		Charges per acre per year on watershed area.		Total charges per acre per year on—	
							Fixed.	Operating.	Fixed.	Operating.	Assessed area.	Watershed area.
					Acres.	*Acres.*						
Big Swan	[1] 400	$20,000	$2,600	$10,790	12,000	15,000	$0.217	$0.900	$0.173	$0.720	$1.117	$0.893
Lacey	185	10,000	1,300	3,200	3,200	3,200	.406	1.000	.406	1.000	1.406	1.406
Pekin-La Marsh	135	7,800	1,000	3,520	2,600	3,200	.385	1.354	.313	1.100	1.739	1.413
Coal Creek	300	20,000	2,600	7,027	6,700	7,426	.388	1.049	.350	.946	1.437	1.296
East Peoria	125	10,500	1,360	1,570	750	1,550	1.810	2.093	.877	1.013	3.903	1.890
Average												1.380

[1] Plant has one 100-horsepower motor and 300 horsepower of steam engine. First cost estimated as if four 100-horsepower motors were installed.

TABLE 17.—*Amounts pumped and unit costs of pumping.*

District.	Year.	Amount pumped.	Average static head.	Cost of lifting 1 acre-foot 1 foot.
		Acre-feet.	*Feet.*	
Louisa-Des Moines	1910	15,900	4.20	$0.086
Do	1911	15,000	2.82	.141
Do	1912	20,132	5.20	.064
Do	1913	16,002	5.74	.063
Coal Creek	1914	6,800	9.40	.110
East Peoria	1914	901	7.02	.158

TABLE 18.—*Probable costs of pumping with typical pumping plants.*

District.	Cost of lifting 1 acre-foot of water 1 foot.	Cost of lifting 1.25 acre-feet 1 foot.	Cost of lifting 1.25 acre-feet—		Fixed charges.	Total cost of pumping per acre per year.	
			5 feet.	10 feet.		Lift 5 feet.	Lift 10 feet.
Louisa-Des Moines	$0.063	$0.079	$0.395	$0.79	$0.406	$0.801	$1.196
Coal Creek	.110	.137	.685	1.37	.35	1.035	1.72
East Peoria	.158	.198	.99	1.98	.877	1.867	2.847

OPERATION AND MAINTENANCE.

It has long been recognized that in the construction of drainage improvements the services of men who make the planning and the supervision of such work a specialty is a business necessity. State drainage laws require that such men be employed on districts organized under the law. It is to be regretted that such laws do not also provide for engineering supervision of the operation of the pumping plant and in the maintenance of all the drainage improvements. The successful and economical operation of the pumping plant and the proper maintenance of the plant and the other drainage improvements require the careful and constant attention of those who are especially fitted by training and experience to do such work. On many districts examined the operating expenses of the pumping plant were unnecessarily large, the drainage improvements were rapidly deteriorating, and large damage had occurred from the partial or complete failure of some of the improvements. Such districts did not have proper supervision.

The present system of having a number of commissioners responsible for the administration of the affairs of the districts is wrong. Much is lost by this division of authority, and responsibility for any feature of the administration is hard to fix. Someone should be hired by the district to have full charge of the administration of the district business, and he should be made entirely responsible for the success of such administration. As an example of the divided authority, the records of one of the districts on the Illinois River showed that in the building of a house for the storage of coal at the pumping plant more money was spent in commissioners' expenses than in the construction of the storage house. An examination of Table 13, showing the costs of the various items for pumping per acre per year, will show that the lowest cost of operation and also of administration is found on the Des Moines County No. 1 District. This district employed engineering supervision and the business of the district was all in the hands of such supervision.

While the need is great for such supervision over all of the drainage improvements it is especially great for the pumping plant. A good pumping plant will often be wasteful, because it is improperly operated. After installation it should be thoroughly tested at various stages of the river, so that the proper operating conditions can be definitely determined. A competent engineer should be placed in charge of the plant, and he should report daily to expert supervision by a complete system of records. From such records it may be possible to devise improved methods of operation or ways of securing greater economy. Without such records it is impossible for anyone to tell in what particulars the operation of the plant is wasteful or is susceptible of improvement. Such a record should show the time of starting and stopping the pumps each day, the heights of the water levels within and without the district, the speeds of the pumps or engines, and the amounts of the boiler feed water and fuel consumed. It should set forth all incidents of importance in the operation of the plant, and may be kept in a form similar to that shown in Table 19. One page should be used for each day. The gauges should be read before the pumps are started, again as soon as the ditch level has reached a fairly steady state, and at every hour during operation. If more than one pump is in operation the speed of each should be recorded. All additional matters should be recorded in the remarks column. An accurate rainfall record should be kept at every plant. The expenses of operating the plant should be carefully kept under the following headings: Fuel; labor; supplies, such as lubricating oil, waste, etc.; repairs; and superintendence. These expenses should be totaled for each month and for the whole season.

TABLE 19.—*Form for daily pumping records.*

Date, ...

Hour.	Ditch gauge.	River gauge.	Lift.	Speed of pump.	Steam pressure.	Rainfall.	Remarks.
	Feet.	*Feet.*	*Feet.*	*R. P. M.*	*Pounds.*	*Inches.*	

The keeping of proper records for a pumping plant is as important as is bookkeeping for a merchant. Failure to do so will mean loss, just as surely in the first case as in the second. Many districts have already suffered from this omission, and unfortunately very few of the districts are profiting by this costly experience. The landowners within the districts should demand of those in charge that such records be kept.

Many districts have followed the shortsighted policy of employing a cheap man to operate the plant during the pumping season and then allowing the plant to be without an engineer for a large part of the year. Such plants have deteriorated very rapidly and are becoming increasingly more expensive to operate. In the end it is much cheaper to employ a competent man by the year, furnish him with a good house, and make conditions attractive enough to hold the same man for a term of years. The plant on the Louisa-Des Moines District has been operated for about five years; it has been in charge of the same man for the entire period. While the salary paid the engineer has been twice the amount that many other districts pay, it has been money well invested, as the plant is now in almost perfect condition and has the appearance of a new plant. Figures already quoted show that the economy of the plant was better in 1913 than it was in 1910. If by careful operation and maintenance the depreciation of such a plant could be decreased from 6 per cent to 5 per cent, a saving of $500 a year would be effected. It is certain that the depreciation on many plants which have been carelessly operated during the pumping season and neglected the remainder of the year is at least 10 per cent. On a plant of this size this difference in rates of depreciation would easily pay the salary of a first-class engineer.

PRESENT STATUS OF DRAINAGE BY PUMPING.

Experience on the Illinois and Mississippi Rivers shows that the total cost of adequate general district drainage improvements, including levees, ditches, and pumping plant, has varied between $20 and $50 an acre. An average figure for all the districts examined on the Illinois and Mississippi Rivers is about $30 per acre. The tendency has been in the last few years to take in the more expensive districts and a considerable increase may be expected in the unit cost of reclamation, particularly on the smaller district. The cost of clearing the land and installing field drainage lies between $5 and $20 per acre. These figures do not include the cost of tile-draining the land, but merely of providing the necessary field ditches. Hence, in some places the total cost of putting the land into profitable cultivation may be as high as $50 per acre. The success that has attended drainage of this character is awakening the interest of those in many parts of the United States who own valley lands subject to overflow;

by profiting by the experience of the districts already under drainage, the new districts will undoubtedly save much money and secure more satisfactory drainage improvements. The best way for one unfamiliar with this kind of drainage to become informed on the subject is to visit and inspect thoroughly a number of the completed districts. Tables 20 and 21 give lists of such districts, with information as to location, size, etc.

SUMMARY.

Numerous tracts in units of from 5,000 to 20,000 acres have been reclaimed along the Illinois River and on both sides of the Mississippi in the States of Illinois, Iowa, and Missouri. In the average district the cost of the general drainage improvements, including levees, ditches, and pumping plant, has been about $30 per acre. This method of reclamation may be expected to be extended constantly to new localities as agricultural land becomes more valuable.

The design and construction of the levees, ditches, and pumping plant require a considerable degree of engineering ability. Poor design may result either in a system so inadequate for its purposes as to render the benefit to the land insufficient to make the undertaking profitable, or it may lead to an expenditure for construction greatly in excess of the amount that could have been made to suffice. In general, the inexperienced tend to underestimate greatly the extent and expense of the work required in such reclamation.

The levees must originally be made of such height and thickness as to afford ample strength and they must also be given careful subsequent attention to secure proper maintenance. The internal drainage ditches should be deep enough to keep the ground-water level at least 3 feet below the surface and their capacity should be sufficient to discharge heavy rains freely to the pumping station. Streams entering the district from higher ground should be diverted around the levees where such a plan is feasible, and provision should be made in such diversion ditches for the collection and storage of silt from hill streams.

The pumping plant should have a capacity sufficient to remove as a minimum amount in 24 hours a quantity of water sufficient to cover the entire district to a depth of 0.3 inch. The capacity should be greater in situations of heavy rainfall and where the run-off of rolling land is received in the district.

The pumping machinery should be so arranged as to reduce to a minimum the work of disposing of the surplus water, and it should be chosen with especial regard to economy and efficiency in operation. Where large fluctuations in the river level are to be expected the machinery must be sufficiently large to operate at the maximum head, and at the same time must be as efficient as practicable for more moderate heads.

Since it has been shown that the average depth of water to be pumped from such districts per year will be about 15 inches, with well-designed and carefully operated plants the total cost per acre of drainage area per year should not exceed 80 cents for a mean lift of 5 feet and $1.20 for a mean lift of 10 feet.

The administration of the business of the district should be placed in the hands of a competent engineer who is familiar with drainage work. He should be held entirely responsible for the proper administration of the district business and especially for the operation and maintenance of the pumping plant. Reliable, skilled attendants should operate the pumping plant under expert supervision and should maintain it in first-class condition the entire year. Full records of the operation of the pumping plant should be kept, as well as detailed classification of expenditures, so that those in charge can determine at all times whether the operation of the plant is as economical as may be possible while securing the results desired from the plant.

Where practicable, gravity outlet sluiceways should be installed in connection with a pumping plant for use during times of low water outside of the drainage district.

The reclamation for agricultural purposes of river bottom lands lying so low that they are subject to serious injury by overflows has been proved by experience in this country and in Europe to be feasible and profitable where the land is of sufficient fertility and where the improvements consist of a system of protective levees supplemented by interior drainage ditches and a suitable pumping plant to remove excessive precipitation which may fall within the district. Such results, however, can come only when the entire plant is properly designed and efficiently maintained and operated.

TABLE 20.—*Summary of pumping districts on Illinois River.*

Name of district.	Name of officer or landowner.	Address.	Area.		Year of first pumping.	Total cost of improvements.	Power.		Pumps.		Engines, number and kind.	Boilers.
			District.	Watershed.			Kind.	Horsepower.	No.	Size.[1]		
			Acres.	*Acres.*						*Inches.*		
Hennepin	Adam Deck	Hennepin	2,600			$130,000	Steam					
Partridge		Chillicothe	3,000									
East Peoria	Eugene Brown	Peoria	750	1,550	1910	45,000	Electric	125	1 1	15 18		
Pekin-La Marsh	C. L. Velde	Pekin	2,600	3,200	1890		do	135	1 1	12 26		
Spring Lake	J. M. Powers	do	12,850	22,500	1909	250,000	Steam	600	1 2	24 48	3 comp. cond. Corliss	Water tube.
Banner-Special	W. R. Curran	do	4,600	8,100	[2] 1915	180,000						
Lacey	L. S. Turner	Havana	3,200	3,200	1900	100,000	Electric	185	1 2	12 20		
Langellier	A. L. Langellier	do	2,000	2,000	1905	70,000	do					
West Mantanzas	John H. Wehner	do	2,700		[2] 1915	134,000	do	175	1 1	18 24		
Otter Creek		do	3,720		1909	107,000	Steam	180	2 1	15 20	3 simple slide valve	Return tubular.
Big Lake	Warren R. Leach	Rushville	3,270	4,300	1914	155,000	Electric	200	2	26		
Coal Creek	Jesse Lowe	Beardstown	6,700	7,425	1898	353,000	do	300	3	20		
Crane Creek	H. V. Teal	Rushville	5,340	6,000	1910	125,000	Steam	250	1	36	Comp. cond. Corliss	Do.
Meredosia	W. F. Roegge	Meredosia	3,760	5,000	1906	135,000	do Electric, 1914	200 100				
Magee Creek	Wade Ervin	Chambersburg	11,500	16,500	1909		Steam	500	2 1	32 36	Simple Corliss Comp. cond. Corliss	Do. Water tube.
Scott County	W. D. Gibbs	Winchester	10,500	11,500	1913	180,000	do	420	2	30	2 simple Corliss	Return tubular.
Big Swan	do	do	12,000	15,000	1906		do Electric, 1913	300 100	3	26		
Hillview	Louis Lowenstein.	White Hall	12,300	18,500	1908		Steam Electric, 1914	250 250	1 1 2	32 26 30	2 comp. cond. vert.	Scotch marine.
Hartwell	Charles Schutz	Hillview	9,000	10,500	[2] 1915		Electric	450	3	30	Comp. cond. vert. slide valve.	300-horsepower return tubular.
Fairbanks	W.D. Fairbanks	Eldred	12,000	15,000	1910		Steam	600	1 1	36 48	Comp. cond. Corliss	450-horsepower water tube.
Eldred	W. M. White	do	9,300	9,500	1911	250,000	do	300	1 1	24 36	Comp. cond. Lentz	Water tube.
Nutwood	Clarence Reddish	Jerseyville	10,700	15,000	1910	275,000	do	500	3	32	Corliss condensing	Return tubular.

[1] All pumps of centrifugal type.

[2] Construction.

TABLE 21.—*Summary of pumping districts on Mississippi River.*

Name of district.	Name of officer or landowner.	Address.	Area.		Year of first pumping.	Total cost of improvements.	Power.		Pumps.		Engines, number and kind.	Boilers.
			District.	Watershed.			Kind.	Horsepower.	No.	Size.[1]		
			Acres.	*Acres.*						*Inches.*		
Drury	C. S. H. Young	Muscatine, Iowa.	5,000	....	1908	$20,000	Steam	250	1	54	Comp. cond. Corliss	Water tube.
Bay Island	M. F. Prouty	New Boston, Ill.	20,000	64,000	1910	242,000	do	800	2	60	do	Do.
Muscatine-Louisa	Sam. Wilson	Wapello, Iowa.	24,000	94,000	[2] 1915	281,000	do	950	1 2	36 54	Uniflow	Water-tube superheat, 125°.
Keithsburg	W. J. Graham	Aledo, Ill	1,400	1,900	1911	52,000	Oil	100	1 1	12 15	....	
Louisa-Des Moines No. 4.	Allen Hall	Oakville, Iowa.	13,000	16,000	1909	108,000	Steam	500	2 1	50 18	Comp. cond. Corliss 3-350 Lentz	Water tube.
Des Moines County No. 1.	J. H. Pettibone	Burlington, Iowa.	28,600	75,000	1911	342,000	do	1,230	1 3	24 54	2 comp. automatic slide valve.	Water-tube superheat, 125°.
Henderson County Nos. 1 and 2.	....	do	17,000	22,400	1914	200,000	do	600	2	48	Comp. cond. Corliss	Water tube.
Lima Lake	John Ensminger	Lima, Ill	13,000	....	1906	105,000	do	400	2	36	Vertical comp. slide valve.	Return tubular.
Indian Grave	J. H. Best	Quincy, Ill	19,000	....	[3] 1914	335,000	....	....	....	....	....	
Fabius	do	do	16,000	16,000	[3] 1914	400,000	....	400	....	....	....	
South Quincy	W. P. Martindale	do	5,500	10,500	[2] 1915	235,000	Electric	400	2	36	....	
Marion County	Ben. Glahn	Palmyra, Mo.	4,000	6,000	1915	135,000	do	200	2	30	....	
South River	L. C. Hendren	Hannibal, Mo.	9,700	11,200	1911	220,000	Steam	200	2	36	Simple Corliss	Do.
Elsberry	....	Elsberry, Mo.	25,000	25,000	[2] 1915	480,000	do	600	2	48	Uniflow	Water-tube superheat.

[1] All pumps of centrifugal type

[2] Construction.

[3] Organized.

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